

Seismic Behavior of RC Shear Walls Strengthened with Fiber-Reinforced Polymer

H. El-Sokkary¹ and K. Galal, M.ASCE²

Abstract: This paper investigates the seismic behavior of RC shear walls strengthened by using carbon fiber-reinforced polymer (CFRP) composites. Three RC shear walls were tested: one control wall and two walls strengthened by fiber-reinforced polymer (FRP), using two different strengthening schemes. The walls were tested when subjected to a constant axial load along with synchronized cyclic moment and shear force at the top of the tested panel. The primary purpose of the FRP retrofit schemes was to increase the flexural and shear capacities of wall panels that experienced higher seismic demands than the design ones. The walls represent the sixth story panel of an eight-story RC, moderately ductile, shear wall designed according to the 2005 National Building Code of Canada. Reported shake table tests on two eight-story walls demonstrated the increase in the demand in higher story levels as a result of the effect of higher modes of vibration, which calls for increasing the capacity of the walls at a high floor level. In this study, the first wall was strengthened by using two layers of unidirectional CFRP sheet that were applied vertically and anchored to the top and bottom slabs, above which unidirectional horizontal C-shaped CFRP sheets were applied. The second wall was strengthened by applying cross-FRP bracings on the two sides of the wall. The three walls were tested under constant axial load and with increasing cycles of synchronized top moment and lateral load with a top moment-to-shear ratio of 2.75 up to failure. The strengthened walls showed satisfactory performance with improved flexural and shear strengths compared to the control wall. The two retrofitted walls showed different load and deformability capacities because of the nature of crack propagation and the orientation of the applied CFRP sheets. DOI: [10.1061/\(ASCE\)CC.1943-5614.0000364](https://doi.org/10.1061/(ASCE)CC.1943-5614.0000364). © 2013 American Society of Civil Engineers.

CE Database subject headings: Reinforced concrete; Rehabilitation; Fiber reinforced polymer; Cyclic tests; Seismic effects; Shear walls.

Author keywords: Reinforced concrete; Retrofit; Carbon fiber-reinforced polymers; Strengthening; Cyclic tests; Seismic performance.

Introduction

In many existing RC buildings, shear walls constitute the seismic force resisting system. Shear walls that were designed according to older design codes may now be seismically deficient according to modern seismic design codes because of their insufficient strength and/or ductility. Shear walls designed according to modern seismic design codes may experience higher demands at upper stories arising from the effects of higher modes of vibrations (Priestley and Amaris 2002; Panneton et al. 2006). The aforementioned situations necessitate upgrading the seismic performance of many existing RC shear walls to meet the requirements of modern seismic design codes. Therefore, there is a need to investigate effective means of retrofitting existing RC structural shear walls to increase their flexural capacity while maintaining capacity and ductility concepts. Such upgrades may be needed at conventional plastic hinge zones at the base of a wall or at higher stories as a result of the unaccounted effects of higher modes of vibration.

Several retrofit techniques for RC shear walls using different materials have been reported in the literature. These ranged from using steel, concrete, fiber-reinforced polymers (FRPs), and shape memory alloys as retrofitting materials, used in different methods of application (Fiorato et al. 1983; Taghdi et al. 2000; Bakis et al. 2002; Vecchio et al. 2002; Effendy et al. 2006). These wall retrofitting techniques aim to improve the strength, stiffness, and ductility, or a combination of these characteristics. The expected mode of failure for a specific existing wall determines the appropriate retrofitting technique. FRP composite materials have received increasing attention in the past few decades as a potential material for retrofitting existing RC structures. FRP laminates, sheets, or rods can be used, and the fibers can be prestressed to increase the efficiency of the retrofit. The flexural capacity of the wall can be increased with FRP sheets by orienting the fibers parallel to the vertical axis of the wall. FRP sheets can be bonded to the wall surface by using epoxy resin and anchored to the wall foundation (or bottom slab) and to the top slab by using steel or FRP anchors. Lombard et al. (2000), Kanakubo et al. (2000), and Antoniadis et al. (2003) tested different details regarding the anchorage of FRP sheets for flexural strengthening of RC shear walls. Burr (2004) and Orton et al. (2008) discussed recent developments in the use of FRP anchors and considerations of their design. Kim and Smith (2009) studied the behavior of handmade FRP anchors under tensile load. The shear strength of the wall can be increased by applying the FRP sheets normal to the axis of the wall to cross potential shear cracks (Paterson and Mitchell 2003; Khalil and Ghobarah 2005). Consequently, both flexural and shear capacities can be increased by applying the fibers in both directions (Lombard et al. 2000) or by using diagonal strips.

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Note. This manuscript was submitted on July 15, 2012; approved on January 16, 2013; published online on January 18, 2013. Discussion period open until March 1, 2014; separate discussions must be submitted for individual papers. This paper is part of the *Journal of Composites for Construction*, Vol. 17, No. 5, October 1, 2013. © ASCE, ISSN 1090-0268/(11)/\$25.00.

Panneton et al. (2006) showed the need for the seismic retrofit of existing code-designed multistory RC shear walls because of the unaccounted effects of higher modes of vibration. Their numerical model indicated that multistory RC shear walls can be susceptible to increased demands at upper floors, demands that were not accounted for in their initial design and detailing because the effects of higher modes of vibration had not been anticipated. To experimentally validate the numerical findings, two eight-story reduced-size shear walls (1:0.429) were constructed and tested under simulated ground motion by using the shake table of the École Polytechnique de Montréal (EPM). The aim was to investigate the effect of higher modes of vibration on multistory RC shear walls subjected to eastern North American ground motions, which are expected to be rich in high frequencies (Ghorbanireneni et al. 2012). Tests on the original walls showed the formation of a plastic hinge at the sixth-story level, in addition to the one at the wall base. These findings indicated that there was a need to retrofit RC walls that are susceptible to increased demands at higher levels. The original walls were rehabilitated by using carbon fiber-reinforced polymer (CFRP) sheets at the two locations that experienced nonlinear response (El-Sokkary et al. 2013). The aim of the rehabilitation scheme for the two walls was to increase the flexural and shear capacities of the wall at the sixth story panel as a result of the observed increase in demand at that level, whereas the base panel was confined by using CFRP sheets to increase the ductility without increasing the flexural strength. The rehabilitated walls were subjected to the same ground motion record applied at the base of the original walls. Although the ground motion records used in the shake table tests reached 200% of the design ground motion, the capacity of the original walls and the FRP retrofitted walls was not reached. Therefore, there was a need to conduct cyclic tests on RC shear wall panels that represent the sixth story panel of the eight-story RC walls to evaluate its full performance up to failure and assess the efficiency of the FRP retrofit schemes.

The objective of this study is to investigate experimentally the effectiveness of using externally bonded CFRP composite sheets to increase the flexural and shear capacities of RC shear walls that are susceptible to increased demands. Three RC shear walls were tested under cyclic loading up to failure. The tested walls represent a control wall and two walls retrofitted with FRP by using two different retrofit schemes. The three walls were tested under constant axial load and with increasing cycles of synchronized top moment and lateral load with a top moment-to-shear ratio of 2.75. The aim was to evaluate the complete performance of the rehabilitated walls up to failure.

Test Setup and Specimens

In this study, instead of testing a complete multistory RC shear wall, only one panel of the wall was tested. The plastic hinge region of a wall or the location that requires retrofitting could be tested by taking into account the effect of the shear wall panels above and below the test panel. The remaining stories were assumed to behave in an elastic manner. Three wall panels were constructed and tested: one control wall panel and two FRP strengthened wall panels using two different retrofit schemes. The concrete compressive strength of the control wall (CW) was 45 MPa, whereas for the FRP-retrofitted walls (RW1 and RW2), it was 37 MPa. Grade 400, 10M deformed steel bars were used as the primary flexural reinforcement and 4.5-mm-diameter plain bars were used for the shear reinforcement and for the hoops. The yield strength of the flexural reinforcement was 450 MPa and the ultimate strength was 550 MPa. The plain bar yield strength was 620 MPa and the

ultimate strength was 720 MPa. To provide confinement of the wall boundary elements, as required by the Canadian Standards Association (CSA) A23.3 (2004) for moderately ductile walls, four unbonded steel bars were provided at the boundary elements and rectangular hoops were spaced at 80-mm intervals. The steel bars were unbonded by using greased sleeves to avoid any contribution to the flexural resistance of the wall panel. The shear reinforcement bars were hooked at their ends to ensure they developed full strength. The wall panel dimensions and reinforcement are shown in Fig. 1. As shown in the figure, a rigid reinforced concrete top block was poured monolithically with the wall and the bottom footing. The top rigid block ensured the uniform transfer of axial load, bending moment, and shear force to the wall section. The bottom rigid block was anchored to the strong floor of the lab. The wall panels were retrofitted by using the Tyfo SCH-11UP composite system (Fyfe Company 2010), comprising unidirectional CFRP sheets and TyfoS epoxy. The FRP anchors used in the retrofit were cut and fabricated from the dry fibers used in the composite system. Table 1 shows the mechanical properties, as provided by the manufacturer.

To simulate the actual state of loads on a panel of an RC shear wall, the three walls were tested under constant axial load and synchronized cyclic top moment and lateral load. Three hydraulic actuators mounted against a steel reaction frame were used, as shown in Fig. 2. Two actuators were placed vertically to allow the application of axial load and in-plane moment at the top of the wall. A horizontal actuator was used to apply the shear force at the top of the wall. A rigid steel I-beam was used to uniformly

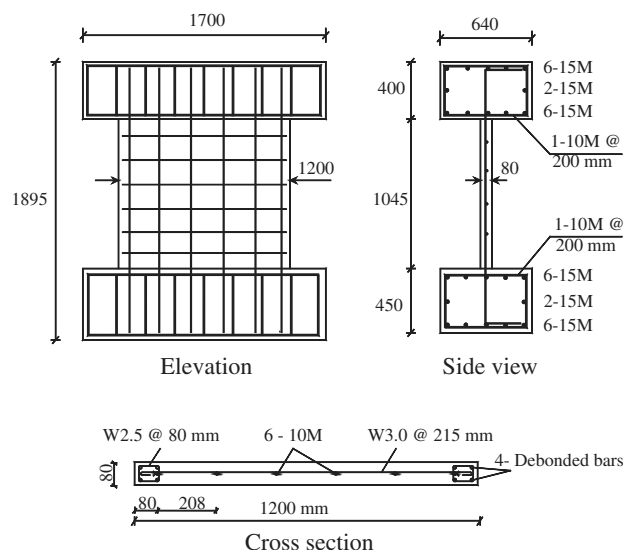


Fig. 1. Wall panel specimen and its reinforcement

Table 1. Mechanical Properties of Tyfo SCH-11UP Composite Used in the FRP Retrofit

Parameter	Typical dry fiber	Epoxy material	CFRP composite	
			Test value	Design value
Tensile strength (MPa)	3,790	72.4	1,062	903
Elongation at break (%)	1.60	5.00	1.05	1.05
Tensile modulus (GPa)	230	3.18	102	86.9
Laminate thickness (mm)	0.175	N/A	0.27	0.27

Note: Data from Fyfe Company (2010).

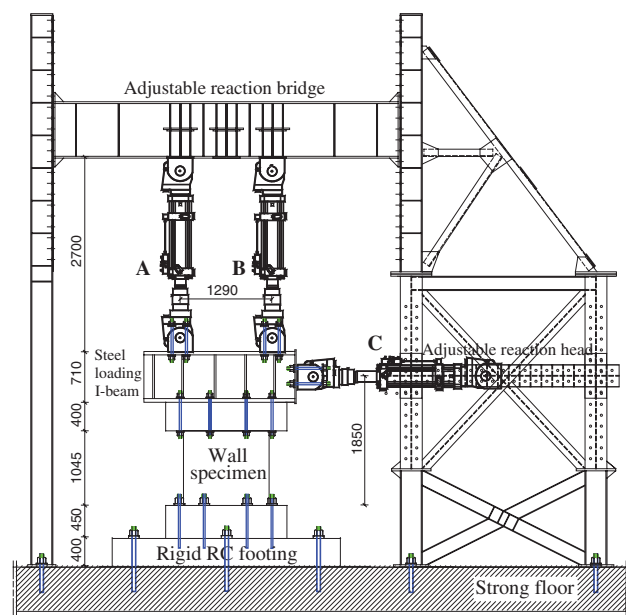


Fig. 2. Test setup for the three walls, tested at the Structures Laboratory of Concordia University

transfer the actuator forces to the top of the wall. Two steel double angle braces connected the rigid I-beam to the laboratory wall. The steel braces were designed to guide the steel loading beam and restrain the out-of-plane movement of the wall panel. The moment-to-shear ratio (M/VL) was kept constant throughout the cyclic tests; this was selected to match the value obtained from the shake table tests on the eight-story walls at the sixth-story level (El-Sokkary et al. 2013). The M/VL at the wall base was selected to be 2.75; therefore, the ratio at the top was equal to 1.88. The selected M/VL classifies the wall as a flexural wall, according to Elnashai et al. (1990). The actuators were controlled to keep the moment value at the wall base equal to 3.3 m multiplied by the wall shear force and a constant axial force of 66 kN at the wall base. This was achieved by controlling the vertical actuators in force control mode based on the feedback from the load cell in the horizontal actuator. The horizontal actuator was controlled in the force mode up to wall yielding; after this, control was switched to a displacement mode. The forces in the two vertical actuators, F_A and F_B (Fig. 2), were related to the horizontal actuator force, F_C , by means of the following equations:

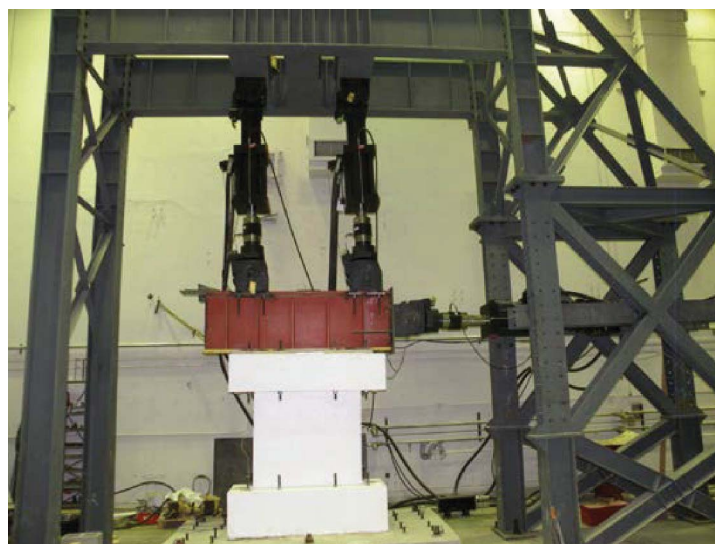
$$F_A = 24 + 1.115F_C \text{ (kN)} \quad (1)$$

$$F_B = 24 - 1.115F_C \text{ (kN)} \quad (2)$$

where the positive sign convention is compression. The equations are valid whether the horizontal actuator is controlled in force or displacement modes. A constant axial load of 48 kN was applied by using both vertical actuators (24 kN per actuator); additionally, the weight of the rigid steel I-beam and the wall top block represented the gravity load carried by the wall panel at the sixth story level during the shake table tests. The coefficients in Eqs. (1) and (2) were calculated to keep the moment at the wall base equal to 3.3 m times the shear force (F_C), whereas the axial force be maintained to be 66 kN.

Control Wall

One CW panel was tested under quasistatic cyclic loading up to failure. The CW panel represented a moderately ductile shear wall,



designed according to the National Building Code of Canada (2005). The flexural capacity of the CW was calculated by using strain compatibility procedures and the concrete and steel properties obtained from the cylinder and coupon tests. The ultimate compressive strain of concrete was assumed to be 0.0035, and the ultimate tensile strength of concrete, f_r , was assumed to be 4.0 MPa. The wall capacity was calculated by taking into account the strain hardening of steel reinforcement. The contribution of compression steel reinforcement to the wall flexural capacity was considered in the calculations. The CW was calculated to have a cracking load of 23 kN, yield load of 39 kN, factored (design) flexural resistance of 47.3 kN, and nominal flexural resistance at failure of 60.8 kN. The nominal shear resistance of the wall was calculated to be 151 kN.

Retrofitted Wall RW1

The primary target of both retrofit schemes was to enhance the seismic performance of the tested wall panels by increasing the flexural capacity of the wall section such that it could resist higher seismic demands. The shake table tests conducted on the eight-story walls (Ghorbanirenani et al. 2012) found that the factored moment at the sixth story level of the tested wall, M_f , was almost 17% greater than the design factored resistance, M_r , when subjected to the design ground motion. Therefore, the retrofit design strategy required that the factored resistance of the retrofitted walls was at least 1.17 times that of the control wall. A value of 1.25 was selected in the design of RW1 and RW2. As a result of increasing the flexural capacity of the wall, the retrofit schemes must consider increasing the shear capacity of the wall panel to maintain the design philosophy of capacity, in which the FRP-retrofitted wall would not fail in shear before reaching its increased flexural capacity.

The first retrofit scheme of RW1 aimed at increasing the flexural capacity of the wall section by applying vertical CFRP sheets at the boundary zones of the wall. This was achieved by applying a 200-mm-wide vertical unidirectional CFRP strip at the wall extremities on both faces, as shown in Fig. 3. The width was designed such that the factored resistance of the retrofitted wall was 1.25 times the factored resistance of the control wall. In the design of the vertical CFRP sheets, the ultimate design strain of the FRP

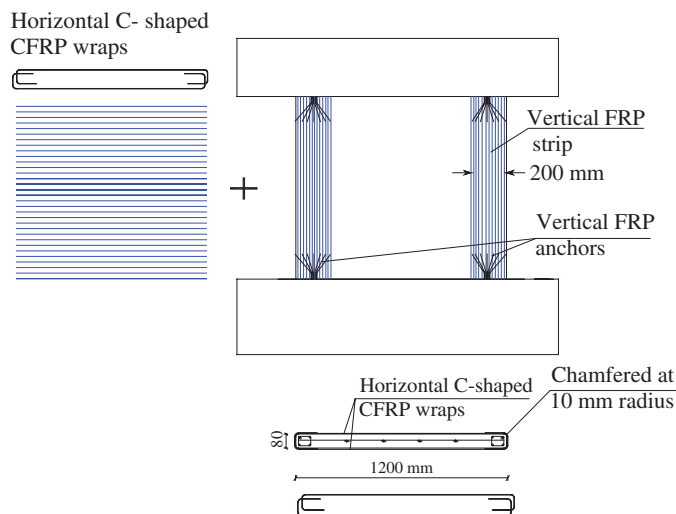


Fig. 3. FRP-retrofitted wall RW1

composite was limited to 0.006, as recommended by Intelligent Sensing for Innovative Structures (ISIS) Canada (2008), to account for any premature anchorage failure or debonding of the CFRP sheets. The ultimate design strain value for the FRP composite was calculated to be 0.00688, according to American Concrete Institute (ACI) 440.2R (2008). A material resistance factor, φ_{FRP} , of 0.75 was used in the design, as recommended by ISIS Canada (2008) for the rehabilitation of flexural members by using CFRP sheets. The retrofitted wall was calculated to have a yield load of 48.5 kN, factored resistance of 60 kN, and nominal resistance at failure of 69.2 kN. The expected failure mode of the retrofitted wall used in the estimation of the ultimate load of the wall was failure of the CFRP vertical sheet system after reaching the design strain. The vertical FRP strips were anchored to the top and bottom blocks by using FRP fan anchors. Two anchors were used for each strip on each wall face at the top and at the bottom. First, the CFRP anchors were securely inserted into the end blocks and the fans were spread on the concrete wall, and then the vertical CFRP strips were applied. Horizontal CFRP sheets were applied on top of the vertical CFRP strips to increase the shear capacity of the wall. Two C-shaped CFRP sheets overlapped at the boundary regions of the wall to provide a better confinement of the wall end columns.

Retrofitted Wall RW2

In this retrofit scheme, the flexural and shear capacities of the wall were increased similarly to the first retrofitted wall RW1, but by using a different layout of the fibers. Instead of using vertical and horizontal CFRP sheets to enhance the flexural and shear behavior of the wall, respectively, diagonal CFRP strips were applied on each face of the wall panel. The 45° diagonal strip results in an inclined force that can be resolved into vertical and horizontal components. These components increase the flexural and shear capacities of the wall section, as shown in Fig. 4. The vertical component of the force is transferred to the top and bottom blocks by using FRP anchors, similar to the first retrofit scheme. The anchors were placed vertically to transfer the vertical component of the force created in the CFRP diagonal sheets. The horizontal component was resisted by applying two 200-mm-wide horizontal C-shaped wraps near the top and bottom blocks of the wall, as shown in the figure. The width of the diagonal strip was selected to be 280 mm. This resulted in an effective cross-sectional area of the inclined fibers (when resolved in the vertical direction), close to that

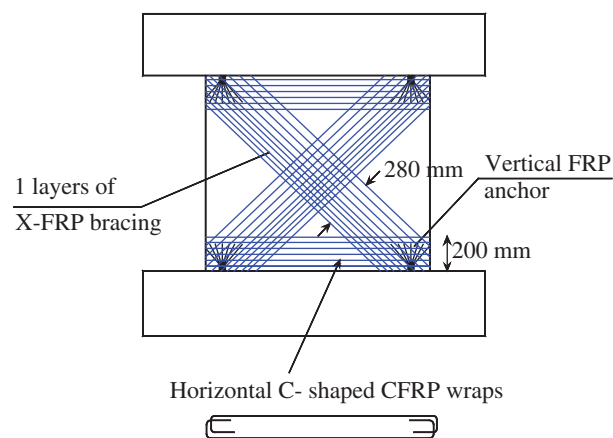


Fig. 4. FRP-retrofitted wall RW2

of the 200-mm-wide vertical strip used in the first retrofit scheme. An advantage to this layout of CFRP sheets is that it makes the wall cracks visible; hence, the retrofitted wall can be monitored after retrofit, which was not the case for the first retrofit scheme in which the whole surface of the wall was covered by the sheets. This retrofit scheme also used less FRP composite material than that of RW1.

Design and Installation of CFRP Anchors

The FRP fan anchors used in the retrofit were prepared from the dry fibers of the CFRP composite system. Each FRP anchor was made of 12 strips; each was 12 mm wide and 680 mm long. The strips were folded into two at midlength and tied at the folded end to form an anchor with an embedment length of 120 mm and a fan length of 220 mm. The diameters of these fabricated anchors ranged from 12 to 14 mm. The anchors were designed to have an ultimate strength equal to 40% of the ultimate strength of the straight fiber composite to account for the strength reduction from the bent fibers. The ultimate anchor capacity (using an ultimate strain of 0.0042 mm/mm) was 33.3 kN. This value matches the coupon tests conducted by Ozbakkaloglu and Saatcioglu (2009), who tested 81 CFRP anchors under direct pullout. Therefore, the capacity of the two FRP anchors used for each vertical or diagonal CFRP strip was equal to 66.6 kN. This value is 15% greater than the ultimate tensile strength of the vertical or diagonal CFRP strips using the typical test value provided by the manufacturer, or 57.8 kN for the strip width selected in this study. This ensured the rupture of the vertical or diagonal CFRP strips before the fracture of the FRP fan anchors. The embedment length of 120 mm exceeded the embedment length used by Ozbakkaloglu and Saatcioglu (2009) in their tests, which was 100 mm to ensure that no bond failure occurred between the CFRP anchor and the concrete substrate. The holes were drilled into the bottom and top blocks, inclined toward the core of the wall from both its faces, at an angle of 20° with the surface of the wall. The drilled holes had a diameter of 20 mm and a depth of 120 mm. For practical applications in existing RC shear walls, hitting the steel rebars is a real concern while drilling the holes for the anchors. Therefore, the location of the steel rebars must be accurately predicted to avoid hitting them during the drilling process.

Instrumentation

The wall specimens were heavily instrumented to measure the applied forces, strains, and displacements. The applied forces were

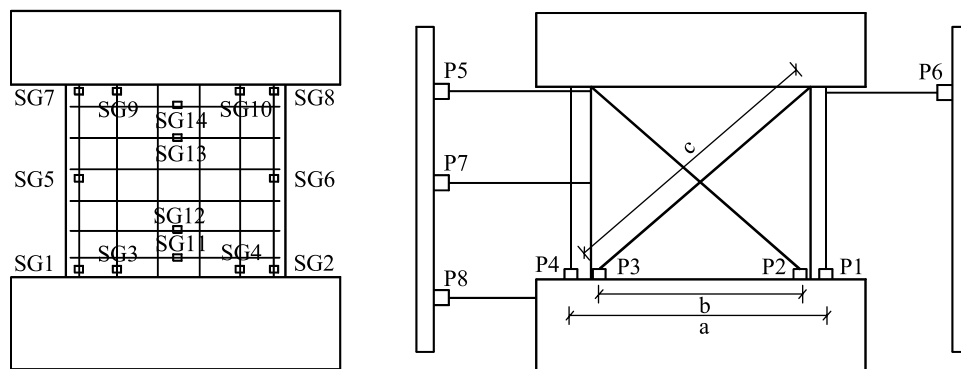


Fig. 5. Strain gauge locations and displacement transducer positions

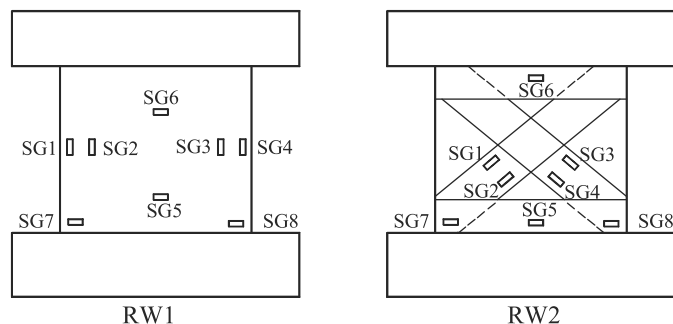


Fig. 6. Locations of the strain gauges installed on the FRP sheets of RW1 and RW2

measured by using the load cells included in the actuators. Fourteen strain gauges were installed on the steel reinforcement of each specimen and another 14 strain gauges were installed on the FRP sheets for each of the retrofitted specimens. The locations of the strain gauges installed on the steel reinforcement are shown in Fig. 5. The locations of strain gauges installed on FRP sheets for RW1 and RW2 are shown in Fig. 6. The wall displacements were measured by using eight cable-extension position transducers (CEPTs) that were attached to the specimen and to two rigid reference walls, as shown in Fig. 5. The top lateral displacement of the wall, the midheight displacement of the wall, the bottom end block movement, the top rotation of the wall, and the wall shear deformation were all measured.

The top rotation of the wall, θ , was obtained by using the readings of the vertical position transducers P1 and P4 according to the following formula:

$$\theta = (P1 - P4)/a \quad (3)$$

Test Results

Control Wall

The CW panel was tested by applying lateral load push and pull cycles at the top of the wall in a force-control mode in three stages: at 20 kN, 30 kN, and at the load corresponding to yielding of flexure reinforcement. After reaching the yielding load, the control mode was switched from force control to displacement control, and the wall was subjected to increasing cyclic displacements that corresponded to displacement ductility levels, μ_{Δ} , of 1.5, 2, 3, 4, and 5, up to failure. The displacement ductility, μ_{Δ} , is defined as the ratio of the displacement level, Δ , to the displacement at yield,

Δ_y . The wall displacement at yield was measured during the test based on the lateral displacement of the wall at the yield strain of the extreme flexural bar. Each cycle started in the push direction and was assigned a positive notation of the lateral force and displacement. The hysteretic relationship between the applied lateral load and the top displacement of the wall is shown in Fig. 7. The first crack was observed near the base of the wall at 20 kN during the pull cycle. Another horizontal crack near the midheight of the wall was observed at a 30-kN load during the pull cycle. Table 2 summarizes the lateral displacements, displacement ductilities, and corresponding capacities for the three tested walls. Fig. 7 shows that after the yielding load, the wall showed a gain in its strength upon increasing the lateral displacement. This gain is primarily attributable to the strain hardening of the flexural steel reinforcement up to a lateral displacement of 4.2 mm ($\mu_{\Delta} = 3.0$) and drift ratio of 0.40%. After the wall yielded, more horizontal fine cracks were observed and began to propagate. These cracks did not widen, whereas it was observed that only the base crack became wider with the increased displacement of the wall. As shown in Fig. 7, the wall did not show an increase in its lateral strength beyond the load cycle at a displacement of 4.2 mm ($\mu_{\Delta} = 3.0$). Concrete crushing was observed at the toe of the wall on the compression side at a displacement ductility $\mu_{\Delta} = 8.0$. At the repeated cycle of the 14-mm-load cycle in the push direction, the extreme flexure reinforcement bar ruptured and the lateral load dropped to +37 kN; i.e., the wall reached its failure limit at this level. At the repeated cycle of the 15.4-mm ($\mu_{\Delta} = 11.0$) load cycle in the pull direction, the other extreme flexure reinforcement bar ruptured and the load dropped to -32.5 kN. The test was stopped after

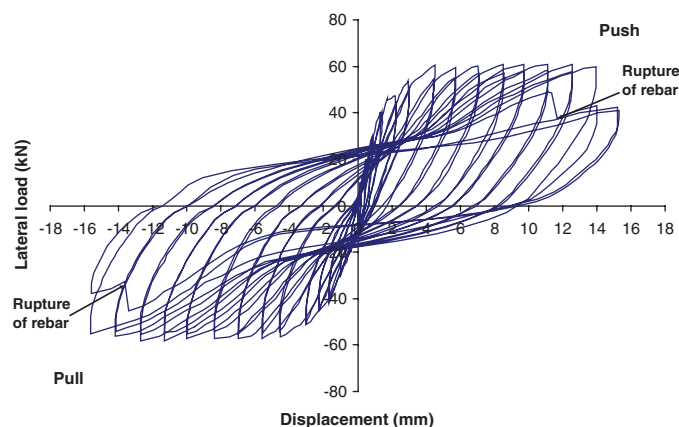


Fig. 7. Lateral load-top displacement relationship of CW

Table 2. Lateral Loads and Deflections of the Three Tested Walls

Wall	V_y (kN)	Δ_y (mm)	Drift at yield (%)	V_u^a (kN)	Δ_u^a (mm)	Drift at ultimate (%)	$\mu_{\Delta u}$	$\Delta_{0.8u}^a$ (mm)	$\mu_{\Delta 0.8u}$
CW	40.5	1.4	0.134	59	14.00	1.34	10.0	14.9	10.6
RW1	59.0	1.5	0.144	106	6.75	0.65	4.5	8.1	5.4
RW2	48.0	1.5	0.144	88	7.50	0.72	5.0	13.5	9.0

^aAverage values for push and pull cycles.

completing the 15.4-mm loading cycle because the wall had reached almost 65% of its lateral load carrying capacity in both the push and pull directions. Minor buckling of vertical bars occurred at the final stage as a result of the exposure of bars after concrete crushing of the wall toes. The failure mechanism of the control wall was the rupture of the extreme flexure reinforcement bars, accompanied by concrete crushing of the wall toes, as shown in Fig. 8.

The top block rotation of the walls, θ , at yield load, at ultimate load, and at 80% of the ultimate load, and the rotational ductility of the walls, are summarized in Table 3 for the three tested walls.

Retrofitted Wall RW1

RW1 was tested by applying lateral loads at the top of the wall of 20 and 40 kN in force control mode to check the proper functioning of the instrumentation. The load was increased in force control mode until the load that corresponded to the yielding of flexure reinforcement was reached. After reaching the yield point, the wall was tested in displacement control mode up to failure. The wall was subjected to cyclic displacements with increasing displacement ductility levels, μ_{Δ} , of 1.33, 1.67, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5, 7.0, 8.0, 9.0, and 10.0. The hysteretic relationship between the applied lateral load and the top displacement of the wall is shown in Fig. 9. Fig. 9 shows that after reaching the yield load, the wall started to gain strength with a relatively high stiffness (compared to CW) upon increasing the cyclic lateral displacement. This type of gain is attributable to the contribution of the vertically anchored FRP strips. At the maximum lateral load level, cracking

of the wall footing near the FRP anchors started to propagate, which marked the beginning of a local footing failure as a result of pullout of the FRP anchors. At a lateral displacement of 7.5 mm ($\mu_{\Delta} = 5.0$), the wall strength started to degrade in both the push and pull directions, and the local cracks in the bottom block of the wall started to widen. Displacements corresponding to 20% strength degradation ($\Delta_{0.8u}$) are usually assumed as an acceptable ultimate performance level (Priestley et al. 1996). The wall was considered to reach its failure capacity at 80% of the ultimate load; however, the test was continued because the wall was able to sustain higher displacement. However, the loading cycle was only applied once after that level. At a lateral displacement of 10.5 mm ($\mu_{\Delta} = 7.0$), the behavior of the wall was similar to the behavior of the control wall, and a complete pullout of the FRP anchors occurred. The bar rupture was delayed as a result of the flexural strength added by the vertical FRP sheets and the presence of FRP anchors in the initial stages. Therefore, there was no steel reinforcement failure before the FRP anchors were fully pulled out. The test was stopped when the wall reached a lateral displacement of 19.5 mm because of the severe damage of the wall footing and before reaching the rupture of extreme vertical steel bars. No rupture of FRP anchors or FRP sheets was observed. The full wrapping of RW1 along its height resulted in relocating the plastic hinge toward the support. The failure mode of RW1 was pullout of the FRP anchors at the wall base, accompanied by a local concrete cone failure of the wall footing, as shown in Fig. 10. Fig. 11 shows the readings of the strain gauges FRP-SG1 and FRP-SG4 installed on the vertical FRP strips at two different locations. As shown in the figures, the vertical FRP strips

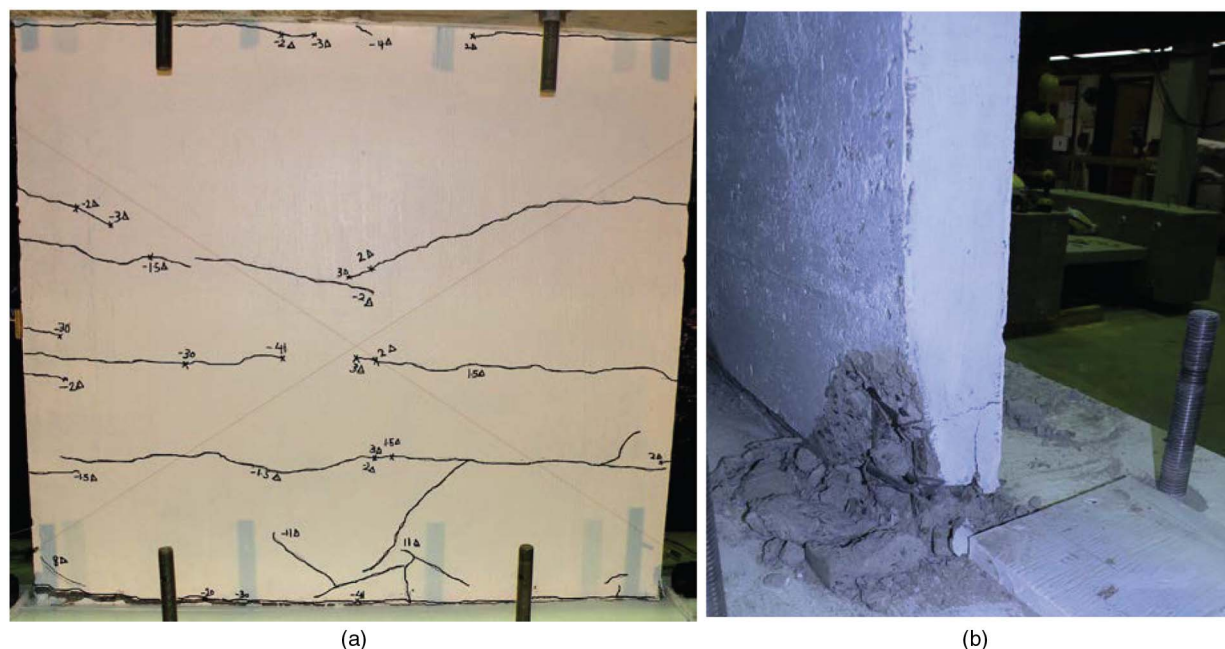
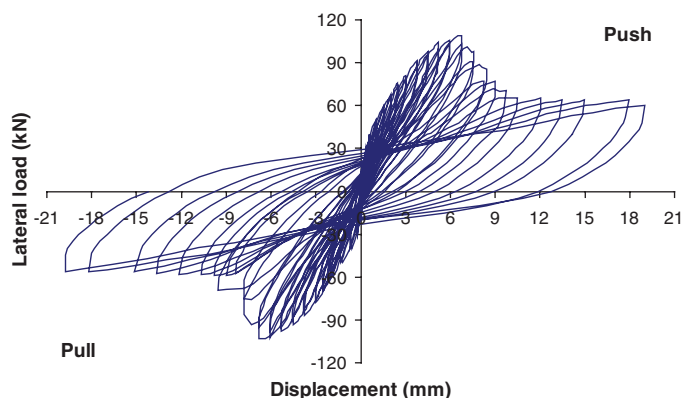


Fig. 8. (a) Crack pattern of CW at failure; (b) concrete toe crushing of CW

Table 3. Top Block Rotation and Rotational Ductility of the Tested Walls

Wall	θ_y	θ_u^a	$\mu_{\theta u}^a$	$\theta_{0.8u}$ (push)	$\mu_{\theta 0.8u}$ (push)	$\theta_{0.8u}$ (pull)	$\mu_{\theta 0.8u}$ (pull)
CW	0.0013	0.0123	9.4	0.0125	9.6	0.0133	10.2
RW1	0.0019	0.0072	3.8	0.0072	3.8	0.0081	4.3
RW2	0.002	0.0088	4.4	0.0132	6.6	0.0194	9.7

^aAverage values for push and pull cycles.

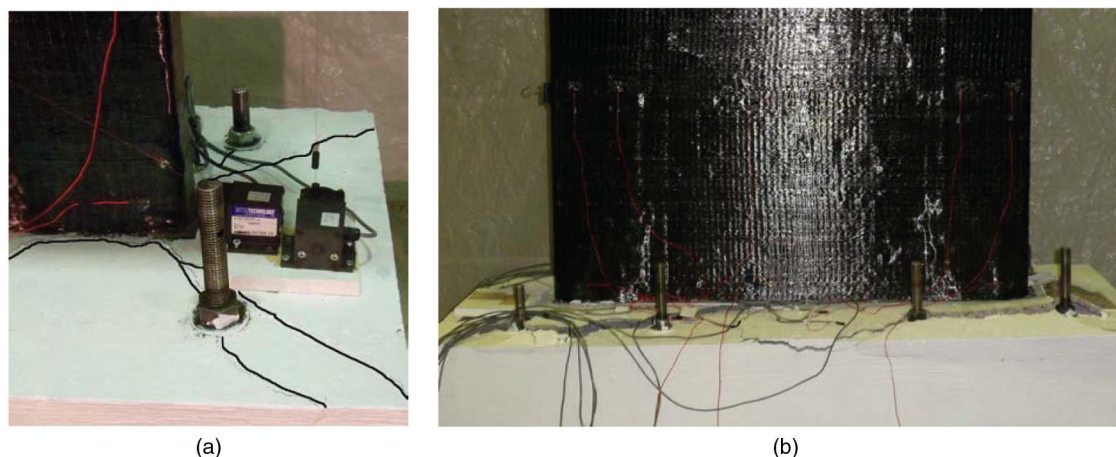
**Fig. 9.** Lateral load-top displacement relationship of RW1

stretched when the wall was pushed in one direction, and the strips were compressed when the load was reversed.

Retrofitted Wall RW2

RW2 was tested by applying cyclic force-controlled lateral load excitations at the top of the wall of 20 and 40 kN, followed by a load cycle to the yield load. After reaching the yield load, the wall was tested in displacement control mode up to failure. The cyclic displacement of the wall was increased to displacement ductility levels, μ_{Δ} , of 1.33, 1.67, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5, 7.0, 8.0, 9.0, and 10.0, similar to RW1. The hysteretic relationship between the applied lateral load and the top displacement of the wall is shown in Fig. 12. This figure shows that after reaching the yield load, the wall continued to gain strength with relatively high stiffness while increasing the lateral displacement as a result of the contribution of the diagonal FRP strips and

the strain hardening of the flexural steel reinforcement. Upon cyclic loading, several cracks developed in the wall and continued to propagate until the wall reached a lateral displacement of 3.75 mm ($\mu_{\Delta} = 2.5$). Upon increasing the cyclic displacement of the wall above this level, no further crack propagation or initiation occurred, but the existing cracks widened, especially the crack just above the bottom CFRP wraps. It is believed that widening of the primary crack above the horizontal CFRP strip and its opening and closure during successive cycles resulted in maintaining a relatively stable lateral load resistance of the wall while increasing its ductility and energy dissipation capacity. At a lateral displacement of 12.0 mm ($\mu_{\Delta} = 8.0$), crushing of the concrete above the well-confined end zones by means of the horizontal CFRP wraps was noticed, and a small portion of the diagonal FRP strip started to rupture. At this displacement level, the strength of the wall started to degrade in the push direction, but the wall was still able to resist more than 80% of its ultimate strength. At a lateral displacement of 13.5 mm ($\mu_{\Delta} = 9.0$), the strength of the wall degraded significantly and more portions of the diagonal FRP strips resisting the pull cycles ruptured. At this displacement ductility level, cracking of the wall footing was observed at the right side of the wall, which indicated the pullout of FRP anchors at that location. The wall was able to sustain a displacement ductility of 10.0 in the pull direction corresponding to a lateral drift ratio of 1.43%. The failure mechanism of RW2 was identified as a rupture of the diagonal FRP strips resisting the pull cycles and pullout of the FRP anchors resisting the push cycles, as shown in Fig. 13. The failure was characterized by concrete crushing above the confined concrete zone wrapped with horizontal CFRP wraps and buckling of the vertical steel reinforcement bars at both sides of the wall. Because the wall was not fully wrapped along its full height, the plastic hinge was able to form above the horizontal FRP wraps near the wall base. This led to crushing of concrete at the wall boundaries, which resulted in a lack of restraint of buckling to the vertical steel reinforcement, as shown in Fig. 13. Fig. 14 shows the readings of FRP-SG2

**Fig. 10.** (a) Local cracking of RW1 footing at the maximum load; (b) failure of RW1

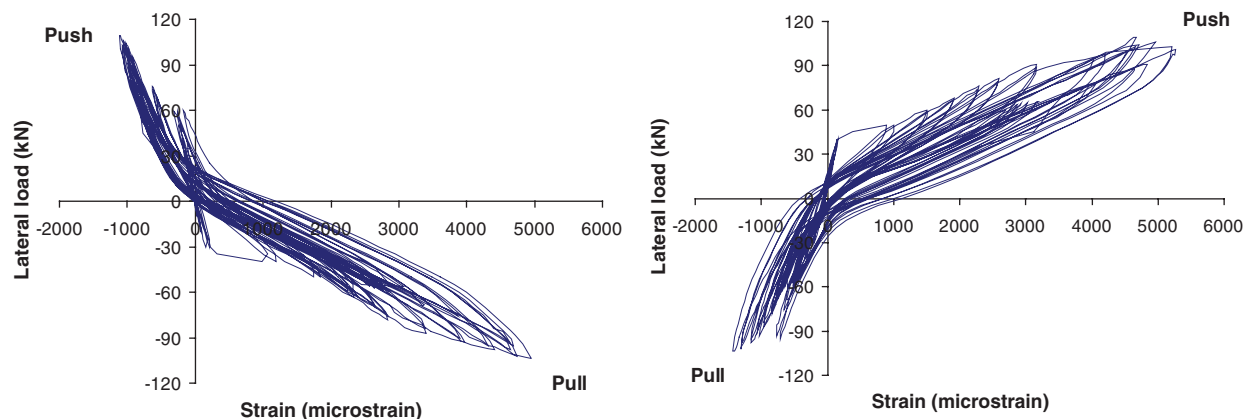


Fig. 11. FRP-SG1 and FRP-SG4 strain readings for RW1

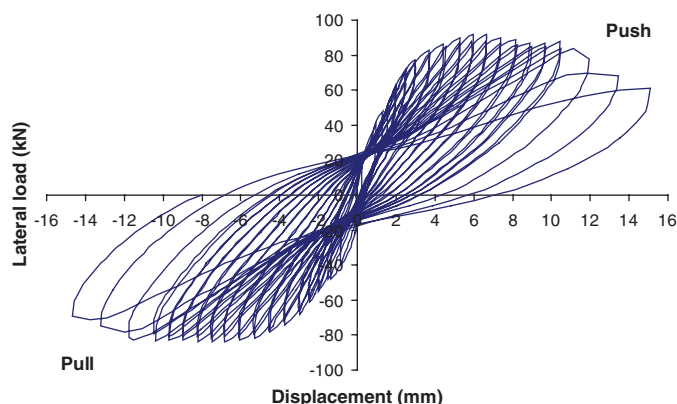


Fig. 12. Lateral load-top displacement relationship of RW2

and FRP-SG4 installed on the diagonal FRP strips at two different locations. As shown in the figures, the diagonal CFRP strips were elongated while the wall was pushed in one direction, and the strips were still elongated with a smaller strain when the load was reversed.

Comparison of Test Results

Envelope of Lateral Load-Drift Relationship

Fig. 15 shows the envelope of the lateral load-drift ratio relationships for the three tested walls. This envelope shows that the control wall was characterized by ductile behavior with an average flexural capacity (for both push and pull cycles) of 59 kN and a drift ratio of 1.47%. The control wall was able to reach μ_{Δ} of 10.0, measured at 20% strength degradation after the peak load. RW1 showed an increase of 80% in the flexural capacity compared to CW, accompanied by a decrease in the displacement ductility of the wall. RW1 reached a displacement ductility of 5.4, measured at 20% strength degradation after the peak load. The yield load was measured at 46% higher than CW at a 7% higher yield displacement. The load-drift relationship for RW1 is the typical relationship expected for any FRP retrofitted element in which the flexural capacity is to be increased. The sudden post-peak strength degradation could arise from the brittle rupture of FRP sheets, debonding, rupture of FRP anchors, or pullout of FRP anchors accompanied by concrete cone failure, which was the case for RW1. The pullout of FRP anchors at the base of RW1 was because the average actual demand

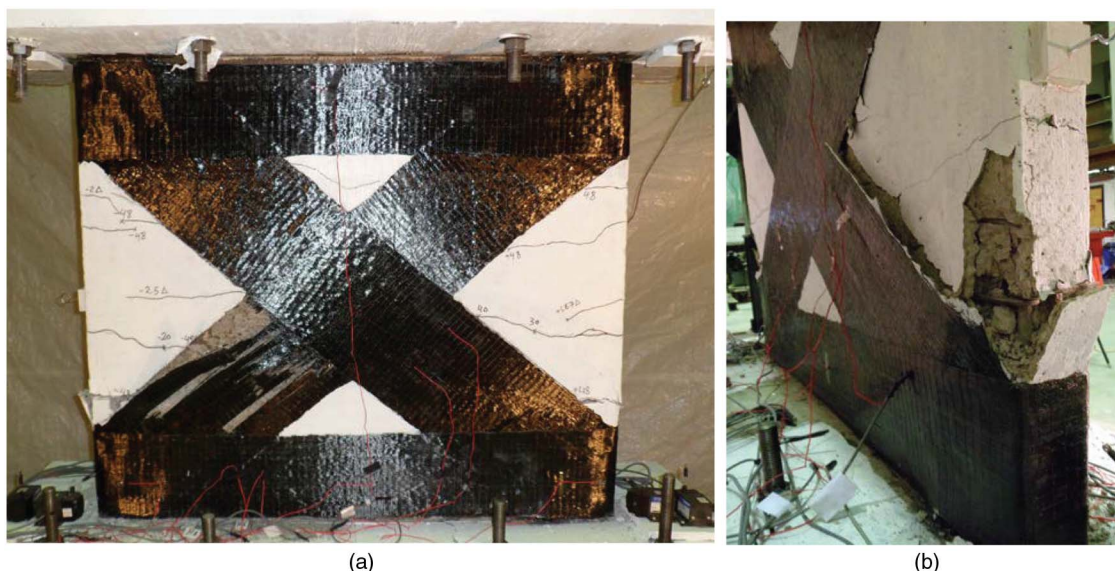


Fig. 13. (a) Rupture of diagonal FRP sheets of RW2; (b) concrete crushing and buckling of flexure reinforcement

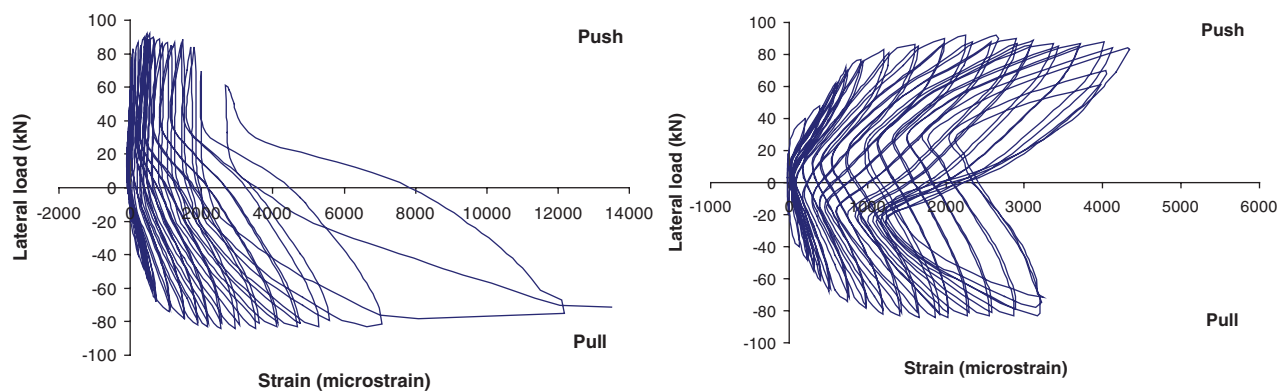


Fig. 14. FRP-SG2 and FRP-SG4 strain readings for RW2

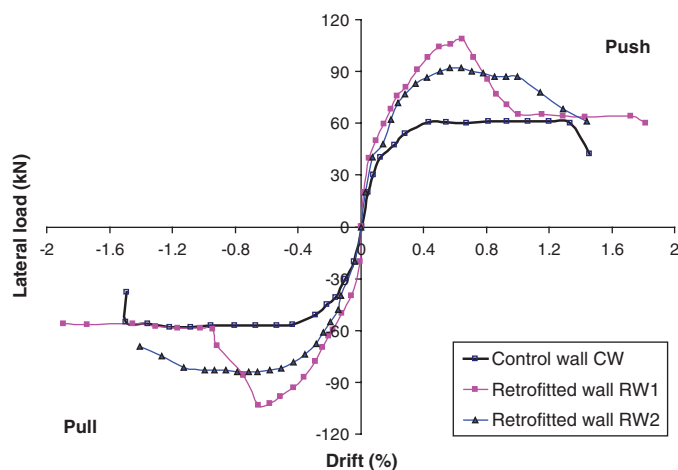


Fig. 15. Envelope for lateral load-drift ratio relationships for the tested walls

on the FRP anchor at the wall base reached almost 37 kN, whereas the expected demand was just 15 kN (based on the target design capacity of the retrofitted wall). This resulted in high shear stresses that exceeded the ultimate shear capacity of the wall footing and caused the concrete cone failure of the footing, followed by pullout of FRP anchors, as shown in Fig. 10. The primary target of the retrofit scheme for RW1 was to increase the flexural capacity of the wall at levels that were subjected to demands higher than those in the design. Therefore, the ductility of the wall is considered to be a secondary parameter that can be overlooked.

RW2 showed a 50% increase of flexural capacity compared to CW, accompanied by similar displacement ductility. RW2 reached a displacement ductility of 9.0, measured at 20% strength degradation after the peak load. The yield load was 19% higher than that of CW at 7% higher yield displacement. RW2 was designed to have the same flexural capacity of RW1. Interestingly, RW2 was found to have less flexural capacity with much higher displacement ductility, shown in Fig. 15 and Table 2. This favorable behavior of RW2 combines the two primary advantages of the other wall panels: the high strength of RW1 and the high ductility of CW. This behavior serves to promote adopting similar retrofit schemes at the plastic hinge regions where higher flexural capacity is required without sacrificing the wall ductility and energy dissipation capacity. It is worth mentioning that the average actual demand on the FRP anchor at the wall base reached almost 27 kN for RW2.

The flexural capacity of RW2 was measured to be less than the capacity of RW1 because both diagonal FRP strips for RW2 were stretched while the wall was pushed (or pulled), as shown from the strain gauge readings in Fig. 14. This is different than the behavior of RW1, in which the vertical FRP strips were stretched on one side of the wall while compressed on the other side, as represented in Fig. 11. This specific behavior of RW2 is attributed to the wide cracks propagated between the top and bottom horizontal CFRP wraps along the height of the wall. These wide cracks did not close completely when the load reversed and the concrete came under compression, which resulted in a residual elongation of the FRP diagonal strips. This phenomenon is explained by using the schematic diagram in Fig. 16. The figure shows the relation between the strains of the diagonal FRP strips during a certain load cycle. Fig. 16 also shows that while the diagonal CFRP Strip A is stretched with a high strain value, ϵ_A , because of the pull cycle, the other diagonal Strip B is also stretched with a smaller strain value, ϵ_B , because of the residual elongation in the fibers caused by the wall cracking. This means that the FRP strips were acting against each other and that the net effective FRP contribution, F_{net} , to the total shear force, F , can be obtained by using the following formulas:

$$F_A = \epsilon_{frpA} E_{frp} A_{frp} \quad (4)$$

$$F_B = \epsilon_{frpB} E_{frp} A_{frp} \quad (5)$$

$$F_{\text{net}} = F_A \cos \theta - F_B \cos \theta = (\epsilon_{frpA} - \epsilon_{frpB}) E_{frp} A_{frp} \cos \theta \quad (6)$$

$$F_{\text{net}} = \epsilon_{\text{net}} E_{frp} A_{frp} \cos \theta \quad (7)$$

where ϵ_{net} = equivalent net strain of the FRP strips, which is equal to the difference between ϵ_{frpA} and ϵ_{frpB} ; E_{frp} = elastic modulus of the fiber composite; A_{frp} = fiber cross-sectional area; and θ = angle between the fiber direction and the horizontal axis.

The maximum average net strain, ϵ_{net} , measured for RW2 at the maximum load prior to the wall strength degradation was 0.004. However, for RW1, the maximum average strain of the vertical CFRP strips was 0.0054. This resulted in a lower flexural capacity of RW2 than RW1, despite the higher FRP strains observed for RW2. The accumulation of permanent strains in the flexure steel reinforcement, the opening of horizontal cracks in RW2 above the horizontal FRP strips, and the ability of the diagonal FRP strips to stretch as the cracks widen allowed RW2 to sustain high lateral displacements while maintaining its strength. On the other hand, the vertical CFRP strips of RW1 controlled the

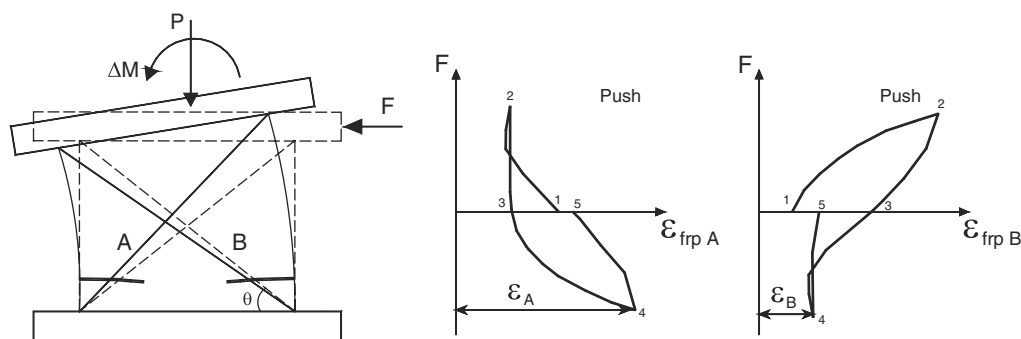


Fig. 16. Strain in the FRP diagonal strips for RW2 at a general load cycle

opening of horizontal cracks of RW1 because the cracks were normal in the fiber direction (observed from the control wall specimen), resulting in a lower displacement ductility capacity of the wall. The maximum average strain of the vertical CFRP strips of RW1 was 37% higher than the average net strain of the diagonal FRP strips for RW2, and was measured at half the displacement ductility of RW2. Having different plastic hinge locations for RW2 yielded to a different flexural lever arm. However, because the tested walls were subjected to a top moment that was almost 44% of the base moment in addition to the shear force, the change in the flexural lever arm in this case will result in a 4.8% change in the wall moment, which can be neglected.

Table 4 shows the experimental and calculated nominal capacities of the retrofitted walls by using the manufacturer mechanical properties. The table shows that the experimental nominal capacities of RW1 and RW2 were higher than those calculated by using section analysis and strain compatibility at the same net strains measured during the tests. The difference between the experimental and predicted capacities implies that the tensile force resisted by the FRP strips was almost 160% higher than that predicted based on the manufacturer data sheet. The different concrete compressive strengths of the tested walls had an insignificant effect on the calculated wall capacities (less than 2%) because the behavior of the wall is primarily governed by flexure.

Envelope of Lateral Load-Top Rotation Relationship

Fig. 17 shows the envelope of the lateral load-top rotation relationships for the three tested walls. Fig. 17 and Table 3 showed that RW2 was able to sustain higher rotation at the top of the wall than CW, whereas RW1 was only able to sustain 65% of the rotation of CW. This indicates that the retrofit scheme used for RW2 was able to improve the overall rotational ductility capacity of the wall while increasing its flexural capacity. Therefore, such a retrofit scheme will be efficient in the retrofit of multistory RC walls at the plastic hinge regions. On the other hand, the retrofit scheme used for RW1 has to be used with caution for cases in which the rotational ductility capacity of the wall is to be maintained.

Table 4. Measured and Calculated Capacities for the Retrofitted Walls

Wall	Experimental V_u (kN) ^a	Experimental FRP net strain (ϵ_{net})	Manufacturer predicted V_u (kN)
RW1	103.0	0.0054	70.0
RW2	88.0	0.004	63.5

^aAverage capacity of push and pull cycles.

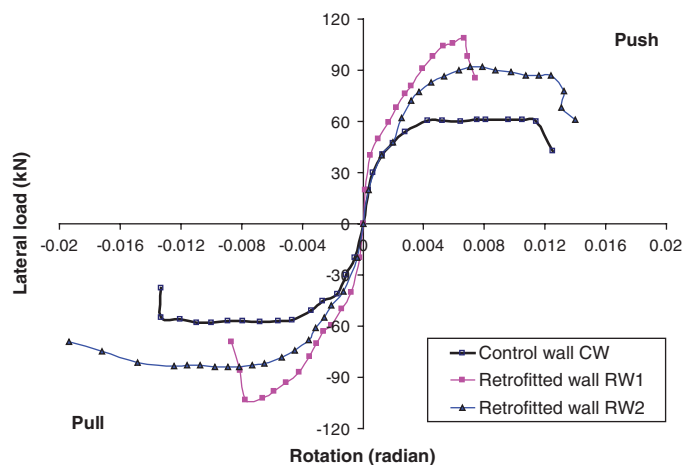


Fig. 17. Envelope for lateral load-top rotation relationships for the tested walls

Conclusions

The seismic behavior of RC shear walls retrofitted by using CFRP was investigated. The experimental program included testing three RC wall panels under lateral cyclic loading up to failure. The walls were subjected to a constant axial load along with synchronized cyclic moment and shear force at the top of each panel. The wall panels represent the control wall and two FRP retrofitted walls using two different retrofit schemes. The primary target of the retrofit schemes was to increase the flexural capacity of the wall section and its shear capacity. The following observations and conclusions were drawn from the results of the experimental study:

- The control specimen was able to sustain lateral displacement ductility, μ_Δ , of 10.0 with no strength deterioration at an average lateral load of 59 kN.
- The lateral load capacities at yield for RW1 and RW2 were approximately 46 and 19% higher than that of CW, respectively, occurring at a 7% higher yield displacement.
- RW1 showed an increase in the flexural capacity of 80%, compared to CW, accompanied by a decrease in the displacement ductility of the wall by approximately 50%.
- RW2 showed an increase in the flexural capacity of 50%, compared to CW, accompanied by a decrease in the displacement ductility of the wall by approximately 15%.
- The higher flexural capacity of RW1 compared to the analytical value based on the FRP mechanical properties provided by the manufacturer resulted in higher demands on the FRP anchors at the wall base, which led to a premature pullout of the anchors.

- The plastic hinge was able to form above the horizontal FRP wraps of RW2 near the wall base because the wall was not fully wrapped along its full height. On the other hand, the full wrapping of RW1 along its height resulted in relocating the plastic hinge toward the support.
- The flexural cracks above the FRP bottom wraps of RW2 did not fully close upon the cyclic load reversals, which resulted in cumulative residual strains in the diagonal FRP strips. This led to a reduced lateral load capacity of RW2 compared to RW1.
- RW2 was able to sustain higher rotation at the top compared to CW, whereas RW1 was only able to sustain 65% of the rotation of CW. Therefore, the retrofit scheme of RW2 is expected to be efficient in the retrofit of multistory RC walls at the plastic hinge regions.

The conclusions are derived based on the findings of the three tested medium-scale walls and more specimens should be tested to generalize the aforementioned conclusions on full-scale walls.

Acknowledgments

The authors are grateful for the financial support received for this study from the Natural Sciences and Engineering Research Council of Canada (NSERC), le Fonds de la Recherche en Québecois sur la Nature et les Technologies (FRQNT) and Centre d'Études Interuniversitaire sur les Structures sous Charges Extrêmes (CEISCE). The authors also acknowledge Fyfe Co. for donating the FRP composite materials used in this research.

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