3rd Specialty Conference on Disaster Prevention and Mitigation
3e Conférence spécialisée sur la prévention et la mitigation des désastres naturels

Montréal, Québec
May 29 to June 1, 2013 / *29 mai au 1 juin 2013*

Cyclic Loading Tests on FRP-Retrofitted RC Shear Walls

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Abstract: The seismic performance of RC shear walls retrofitted using FRP composites is investigated. Three RC walls were tested; one control wall and two FRP-retrofitted walls were tested under cyclic lateral loads that represent flexure, axial and shear forces applied at the top of the wall. The wall specimens represent the 6^{th} storey panel of an 8-storey RC wall designed according to the National Building Code of Canada. The main purpose of the two FRP-retrofit schemes is to increase the flexural and shear capacities of the tested wall and to assess the effectiveness of the FRP-retrofit scheme up to failure. The first wall was strengthened using uni-directional vertical FRP sheet that were anchored to the top and bottom slabs, above which uni-directional horizontal FRP wraps were applied. The second wall was strengthened by applying diagonal FRP-anchored strips on the two sides of the wall. The enhancement in the seismic performance of the walls was evaluated.

1 Introduction

Many existing RC buildings use shear walls as their seismic force resisting system. Some of these shear walls were designed according to older design codes and are currently deemed to be seismically deficient according to new seismic design codes due to their insufficient strength and/or ductility capacities. The aforementioned situations necessitate upgrading the seismic performance of many existing RC shear walls to meet the requirements of modern seismic design codes. As such, there would be a need to retrofit existing RC shear walls to increase their capacity at locations of higher seismic demands. These could be at the plastic hinge zone at the base of the wall, or at higher stories due to the effects of higher modes of vibration (Tremblay et al. 2001).

Different retrofit techniques of RC shear walls using different materials were reported in the literature. These ranged from using steel, concrete, fibre-reinforced polymers, and shape memory alloys as retrofitting materials used in different methods of application. These retrofitting techniques aim to improve the wall strength, stiffness, ductility, or a combination of these. Fibre-reinforced polymer (FRP) composite materials have received an increasing attention in the past few decades as a potential material for retrofitting of existing RC structures due to their high strength, light weight, ease of application, and their high resistance to corrosion. FRP laminates, sheets or rods can be used, and the fibres might be prestressed to increase the efficiency of retrofit. The wall flexural capacity can be enhanced by orienting the fibres parallel to the wall axis. FRP sheets are bonded to the wall surface using epoxy resin and anchored to the wall foundation and to the top slab using steel or FRP anchors. Lombard et al. (2000), Kanakubo et al. (2000) and Antoniades et al. (2005) discussed several ways of anchorage of FRP sheets that can be used for flexural strengthening. Additional shear strength contribution can be obtained by orienting the fibres normal to the axis of the wall to cross potential shear cracks (e.g. Paterson and Mitchell 2003, and Khalil and Ghobarah 2005). Both flexural and shear capacities can be also enhanced by applying the fibres in both directions (Lombard et al. 2000) or by using diagonal strips. The objective of this study is to investigate experimentally the effectiveness of externally bonded carbon fibre-reinforced polymer (CFRP) composite sheets in increasing the flexural and shear capacities of RC shear walls that are susceptible to increased demands. Three RC shear wall panels were tested under cyclic loading up to failure. The tested walls represent a control wall and two FRP-retrofitted walls using two different retrofit schemes.

2 Experimental program

2.1 Test specimens and setup

Three walls were constructed and tested. The test walls represent the 6th storey panel of the 8-storey walls that experienced higher demands than those stated in the design code due to higher mode effects (Ghorbanirenani et al. 2010). The walls were designed according to the NBCC (2005) and CSA-A23.3 (2004) as moderately ductile walls with ductility-related reduction factor, R_d, of 2.0 and overstrength-related reduction factor, R_o, of 1.4. The control wall CW and the two retrofitted ones (RW1 and RW2) were constructed using ready mix concrete of characteristic compressive strength of 45 and 37 MPa, respectively. Grade 400, 10M deformed steel bars were used as the main flexural reinforcement and 4.5 mm diameter plain bars were used for the shear reinforcement as well as the hoops. The flexure steel yield strength was measured in average to be 450 MPa, its ultimate strength was 550 MPa, the plain bar yield strength was 620 MPa, and its ultimate strength was measured to be 720 MPa. In order to provide confinement of the wall boundary elements as required by 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 intentionally unbonded in order not to contribute to the flexural resistance of the wall panel. The wall dimensions and reinforcement are shown in Figure 1. As shown in the figure, a rigid RC top block was poured monolithically with the wall and the bottom footing. The top rigid block ensures the uniform transfer of axial load, bending moment and shear force to the wall section. Static cyclic loading procedures were applied to study the behaviour of the tested walls under lateral seismic forces. The test setup consists of three MTS hydraulic actuators which are mounted against a steel reaction frame as shown in Figure 2. The two vertical actuators were used to apply an axial compression force and a moment, whereas the horizontal actuator was used to apply a horizontal shear force (that resulted in an additional moment at the base of the wall panel). A rigid steel I-beam was used to transfer the actuator forces to the wall top block uniformly.

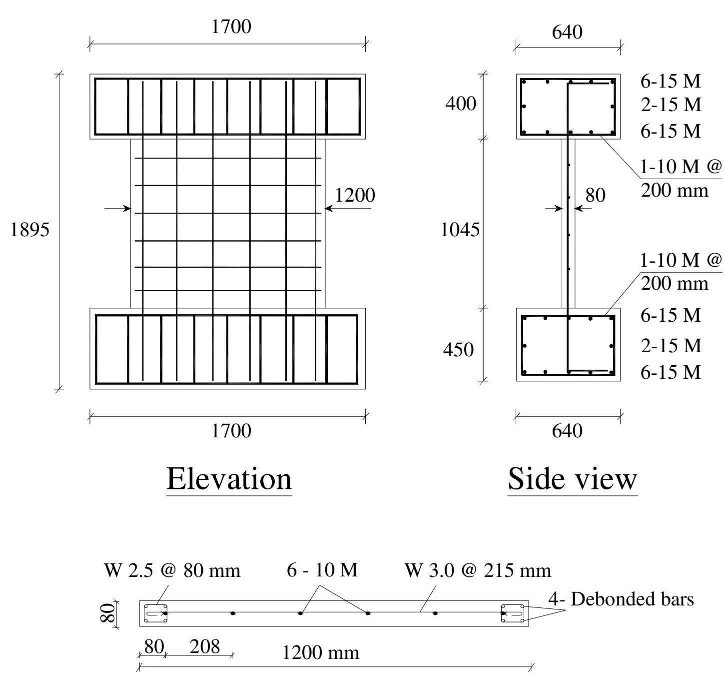

Figure 1. The wall specimen and its reinforcement.

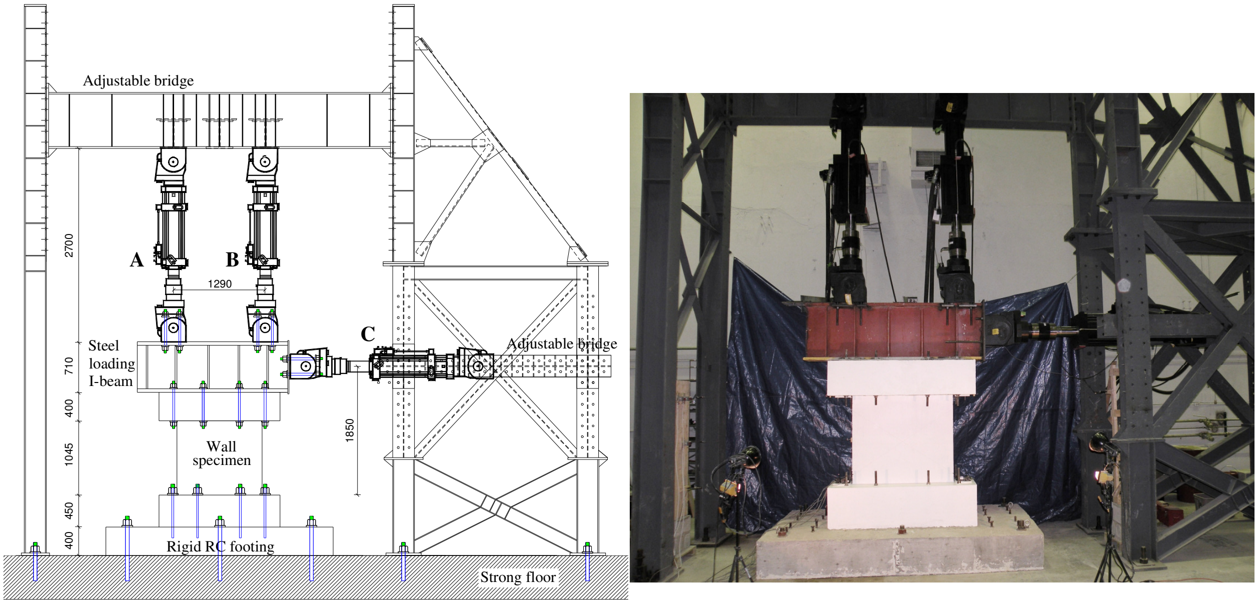

Figure 2. Test setup of the three walls.

Two perpendicular double angle steel braces were connecting the rigid I-beam to the laboratory wall. The steel braces were designed with slots that would guide the steel loading beam and allow a smooth in-plane movement of the wall, yet they would eliminate any out-of-plane movement that may arise from misalignment of the horizontal force or due to possible unsymmetrical damage of the wall at failure. The moment-to-shear ratio (M/VL) at the wall base was selected to be 2.75 and therefore, the ratio at the top was equal to 1.88. The selected M/VL ratio 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 times the wall shear force, in addition to the constant axial force of 66 kN at the wall base. This was achieved by controlling the vertical actuators in force control based on the feedback from the load cell in the horizontal actuator. The horizontal actuator is controlled in the force mode up to the wall yielding load; afterwards, the control mode is switched to the displacement mode. The forces in the two vertical actuators F_A and F_B (Figure 2) are related to the horizontal actuator force F_C using the following equations:

$$F_A = 24 + 1.115\, F_C \text{ (kN)} \tag{1}$$
$$F_B = 24 - 1.115\, F_C \text{ (kN)} \tag{2}$$

where the positive sign convention is compression. The equations are valid whether the horizontal actuator is controlled in a force or displacement mode. A constant axial load of 48 kN was applied using both vertical actuators (24 kN per actuator) which represents the gravity load carried by the wall panel at the 6th storey level.

Carbon fibre-reinforced polymer (CFRP) composites were used for the retrofit of the wall panels. Tyfo® SCH-11UP composite system (Fyfe 2010) with uni-directional CFRP sheets was used for both retrofitted walls. The resin material Tyfo S epoxy was used as recommended by the manufacturer. The FRP anchors used in the retrofit were cut and fabricated from the dry fibres used in the Tyfo SCH-11UP composite system. Total of 16 anchors were used for each of the retrofitted wall specimens. Table 1 shows the mechanical properties of the Tyfo® SCH-11UP composite system; dry fibre, TyfoS epoxy, and CFRP composite (Fyfe 2010) used in the retrofit process.

Table 1. Mechanical properties of Tyfo® SCH-11UP Composite used in the FRP-retrofit (Fyfe 2010).

Parameter	(a) Typical dry fibre	(b) Epoxy material	(c) CFRP composite	
			Test value	Design value
Tensile strength (MPa)	3790	72.4	1062	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	NA	0.27	

2.2 Control wall

One control wall CW was tested under static cyclic loading up to failure. The control wall represents the 6th storey panel of the 8-storey wall tested under axial, top moment, and lateral load excitation. The flexural capacity of the control wall was calculated using the strain compatibility procedures and using the concrete and steel properties obtained from the cylinder and coupon tests. The concrete ultimate compressive strain was assumed to be 0.0035, and the concrete ultimate tensile strength f_r was taken 4.0 MPa. The wall capacity was calculated 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 control wall was calculated to have a cracking load of 23 kN, yield load of 39 kN, factored flexural resistance of 47.3 kN and nominal flexural resistance at failure of 60.8 kN. The wall nominal shear resistance was calculated to be 151 kN.

2.3 First retrofit scheme for RW1

The main target of both retrofit schemes was to enhance the seismic performance of the tested walls by increasing the flexural capacity of the wall section in order to be able to resist the higher demands at the top floors of multi-storey shear walls arising from the higher mode effects (El-Sokkary et al. 2013). From the shake table tests conducted on the 8-storey walls, it was found that the factored moment at the 6th storey level of the tested wall when subjected to the design ground motion M_f was almost 17% greater than the design factored resistance M_r. Therefore, the retrofit design strategy requires that the factored resistance of the retrofitted walls would be at least 1.17 times that of the control wall. A value of 1.25 was selected in the design of the retrofitted walls RW1 and RW2. As a result of increasing the wall's flexural capacity, the retrofit schemes must consider increasing the shear capacity of the wall panel to continue following the capacity design philosophy, where the FRP-retrofitted wall would not fail in shear before reaching its increased flexural capacity.

The first retrofit scheme of RW1 aimed to increase the flexural capacity of wall section by applying vertical CFRP sheets at the boundary zones of the wall. This was achieved by applying a 200 mm wide vertical uni-directional CFRP strip at the wall extremities on both faces as shown in Figure 3. The chosen width was designed so that the factored resistance of the retrofitted wall would be 1.25 times the factored resistance of the control wall. In the design of the vertical CFRP sheets, the ultimate strain of the FRP composite was limited to 0.006 as recommended by ISIS Canada (2008) to account for any premature anchorage failure or debonding of the CFRP sheets. A material resistance factor φ_{FRP} of 0.75 was used in design as recommended by ISIS Canada (2008) for rehabilitation of flexural members using carbon FRP 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 wall's ultimate load 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 using FRP fan anchors. Two anchors were used for each strip on each wall face at the top and the bottom. On top of the vertical CFRP strips, horizontal CFRP sheets were applied to increase the wall shear capacity. Two C-shaped CFRP sheets overlapped at the boundary regions of the wall to provide a better confinement of the wall end columns.

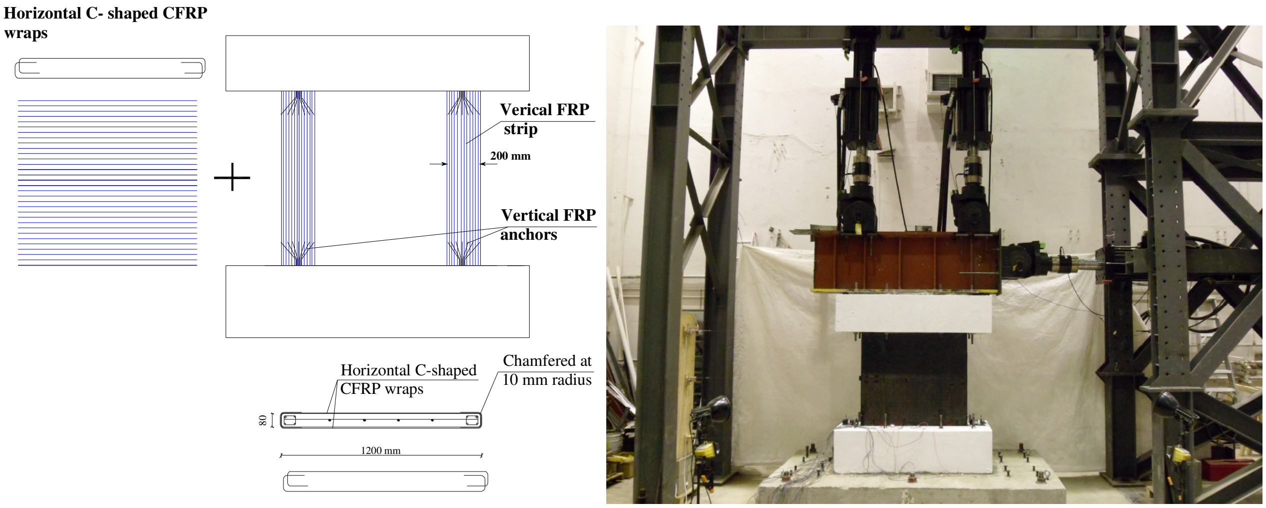

Figure 3. FRP-retrofitted wall RW1.

2.4. Second retrofit scheme for RW2

In this retrofit scheme, the flexural and shear capacities of the wall were increased similar to the first retrofitted wall RW1, but using a different layout of the fibres. Instead of using vertical and horizontal CFRP sheets to enhance the flexural and shear behaviour of the wall, respectively, diagonal CFRP strips were applied on each face of the wall panel. The 45º diagonal strip would result in an inclined force that will be resolved into vertical and horizontal components. These components would increase the flexural and shear capacities of the wall section as shown in Figure 4. The vertical component of the force will be transferred to the top and bottom blocks 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 will be resisted by applying two 200 mm wide horizontal C-shaped wraps near the wall top and bottom blocks as shown in the figure. The width of the diagonal strip was selected to be 280 mm. This will result in an effective cross sectional area of the inclined fibres (when resolved in the vertical direction) close to that of the 200 mm wide vertical strip used in the first retrofit scheme. This layout of the CFRP sheets will make the wall cracks visible and hence the retrofitted wall can be monitored after retrofit which was not the case for the first retrofit scheme where the whole wall surface was covered by the sheets.

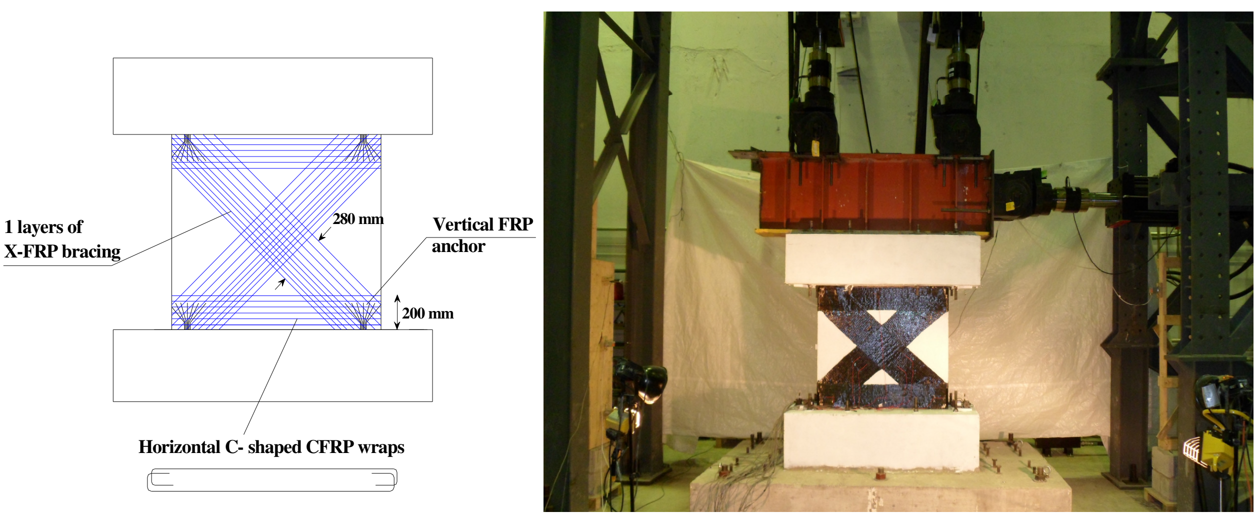

Figure 4. FRP-retrofitted wall RW2.

3 TEST RESULTS

3.1 Control wall CW

The hysteretic relationship between the applied lateral load and the wall top displacement is shown in Figure 5. The yield load occurred at 40.5 kN with a lateral displacement of 1.4 mm corresponding to a lateral drift ratio of 0.134%. From Figure 5, it can be seen that after the yielding load, the wall showed a gain in its strength upon increasing the lateral displacement. This is mainly due to the strain hardening of flexural steel reinforcement up to a lateral displacement of 4.2 mm (μ_Δ = 3.0) and drift ratio of 0.40%. After the wall yielding, more horizontal fine cracks were observed, and they began to propagate. These cracks did not widen, whereas it was observed that only the base crack becomes wider with the increased displacement of the wall. As can be seen from Figure 5, the wall did not show an increase in its lateral strength beyond the load cycle at displacement of 4.2 mm (μ_Δ = 3.0). The ultimate strength measured for the control wall at that displacement level was +61 kN in push direction, and -57 kN in pull direction. Concrete crushing was observed at the toe of the wall at the compression side at a lateral displacement of 11.2 mm, which corresponds to μ_Δ = 8.0 and a drift ratio of 1.08%. The control specimen was able to sustain a lateral displacement of 14 mm, which corresponds to μ_Δ = 10.0 and a drift ratio of 1.34%, without any strength deterioration. At the repeated cycle of the 14 mm load cycle in 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 (μ_Δ = 11.0) load cycle in pull direction, the other extreme flexure reinforcement bar ruptured and the load dropped to -32.5 kN. The test was stopped after completing the 15.4 mm loading cycle as the wall reached almost 65% of its capacity in both push and pull directions. The maximum lateral drift that the control wall reached before failure is 1.34% at 14 mm lateral displacement, which corresponds a displacement ductility μ_Δ = 10.0. The failure mechanism of the control wall was rupture of the extreme flexure reinforcement bars accompanied by concrete crushing of the wall toes.

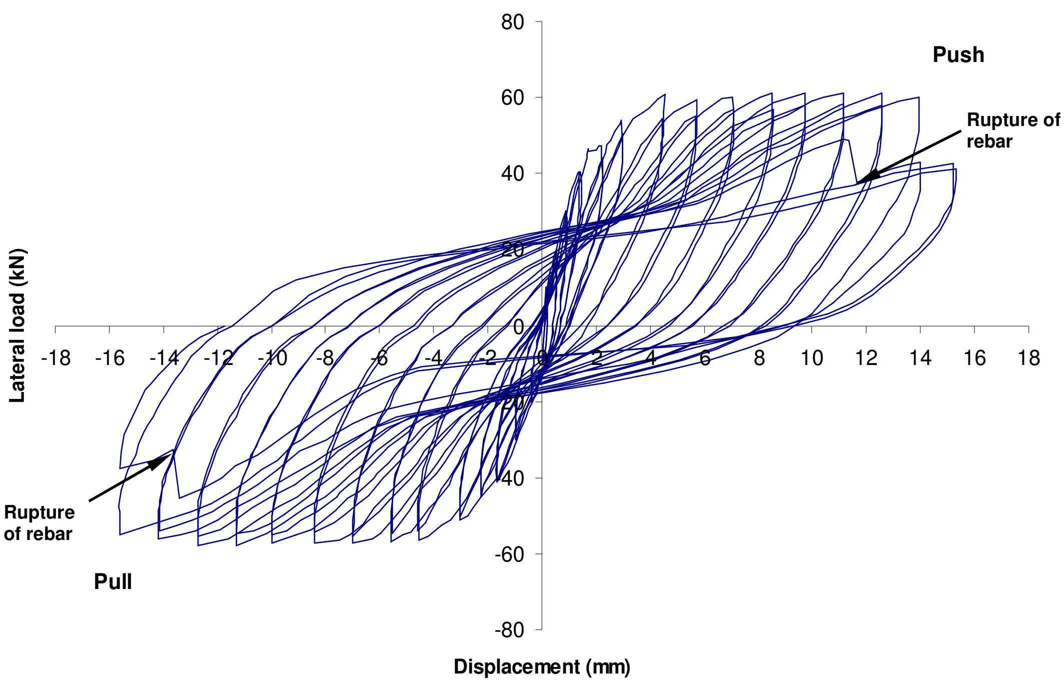

Figure 5. Lateral load-Top displacement relationship of the control wall CW.

3.2. First retrofitted wall RW1

The hysteretic relationship between the applied lateral load and the wall's top displacement is shown in Figure 6. The yield load was determined to be 59 kN, occurring at a lateral displacement of 1.5 mm which corresponds to a lateral drift ratio of 0.144 %. From Figure 6, it can be seen that after the yield load, the wall started to gain strength with a relatively high stiffness (as compared to the control wall CW) upon increasing the cyclic lateral displacement. This is mainly attributed to the contribution of the vertically anchored FRP strips. The retrofitted wall RW1 was able to reach a lateral load of +109 kN in push direction and -103 kN in pull direction at a lateral displacement of 6.75 mm, corresponding to μ_Δ = 4.5 and lateral drift of 0.65%. At the maximum lateral load level (109 kN), cracking of the wall footing near the FRP anchors started to propagate at this high level of force, which marked the beginning of a local footing failure due to pull out of FRP anchors. At a lateral displacement of 7.5 mm (μ_Δ = 5.0), the wall strength started to degrade in both push and pull directions, and the local cracks in the wall's bottom block were becoming wider. Displacements corresponding to 20% strength degradation ($\Delta_{0.8u}$) are usually taken as an acceptable ultimate performance level (Priestley et al. 1996). At a displacement ductility of 5.5, the wall strength degraded to 78% of the wall ultimate strength in push direction and 75% in pull direction which can be identified as the wall's failure displacement ductility level at a drift ratio of 0.79%. The wall was considered to reach its failure capacity at this level, yet the test was continued as the wall was able to sustain higher displacement, but the loading cycle was only applied once after that level. At a lateral displacement of 9.0 mm (μ_Δ = 6.0), the strength of the retrofitted wall WR1 reached almost that of the control wall in the pull direction. At a lateral displacement of 10.5 mm (μ_Δ = 7.0), the wall behaviour was similar to the control wall behaviour and a complete pull out of the FRP anchors occurred. The test was stopped when the wall reached a lateral displacement of 19.5 mm due to the severe damage of the wall footing. No rupture or debonding of FRP anchors or FRP sheets was observed. The failure mode of the retrofitted wall RW1 was pull out of FRP anchors at the wall base accompanied by a local concrete cone failure of the wall footing.

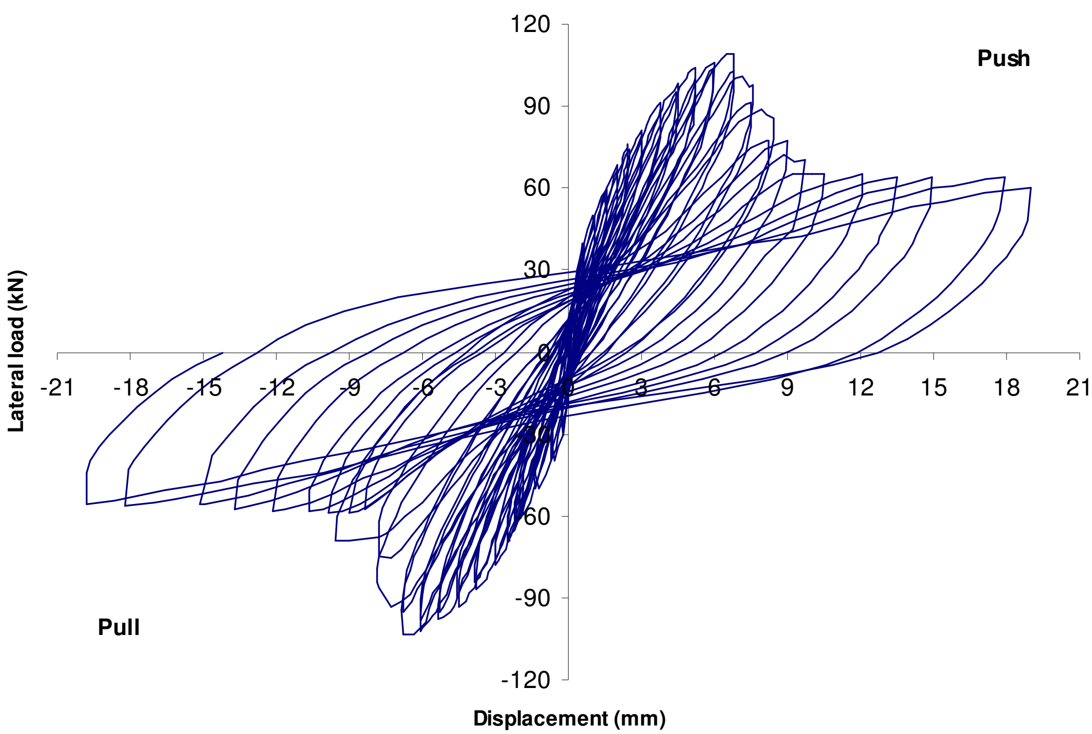

Figure 6. Lateral load-Top displacement relationship of the retrofitted wall RW1.

3.3 Second retrofitted wall RW2

The hysteretic relationship between the applied lateral load and the wall's top displacement is shown in Figure 7. The load at yielding of flexure reinforcement was 48 kN at a lateral displacement of 1.5 mm, which corresponds to a lateral drift ratio of 0.144%. From Figure 7, it can be seen that after reaching the yield load, the wall continued to gain strength with relatively high stiffness while increasing the lateral displacement due to the contribution of the diagonal FRP strips as well as the strain hardening of the flexural steel reinforcement. Upon cyclic loading, several cracks were developed in the wall and they continued to propagate until the wall reached a lateral displacement of 3.75 mm (μ_Δ = 2.5). Upon increasing the wall cyclic displacement above this level, no more crack propagation or initiation occurred, yet it was observed that the existing cracks get widened especially the crack just above the bottom CFRP wraps. It is believed that widening of the main 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. The retrofitted wall was able to resist a lateral load of +92 kN in the push direction and -84 kN in the pull direction at a lateral displacement of 6.75 mm, corresponding to a displacement ductility μ_Δ = 4.5 and a lateral drift of 0.65%. At a lateral displacement of 12.0 mm (μ_Δ = 8.0), crushing of the concrete above the well confined concrete 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 wall strength started to degrade in the push direction but the wall was still able to resist more than 80% of its ultimate strength. Therefore, the wall was able to sustain a displacement ductility of 8.0 in the push direction corresponding to a lateral drift ratio of 1.15%. At a lateral displacement of 13.5 mm (μ_Δ = 9.0), the wall strength degraded significantly and more portions of the diagonal FRP strips resisting the pull cycles were rupturing. At this displacement ductility level, cracking of the wall footing was observed at the right side of the wall which indicates the pull out of FRP anchors at that locationThe wall was able to sustain a displacement ductility of 10.0 in pull direction corresponding to a lateral drift ratio of 1.43%. The failure mechanism of the retrofitted wall RW2 was identified as rupture of the diagonal FRP strips resisting the pull cycles, and pull out of the FRP anchors resisting the push cycles. The failure was accompanied by concrete crushing above the confined concrete zone wrapped with horizontal CFRP wraps and buckling of the steel reinforcement bars at both wall sides.

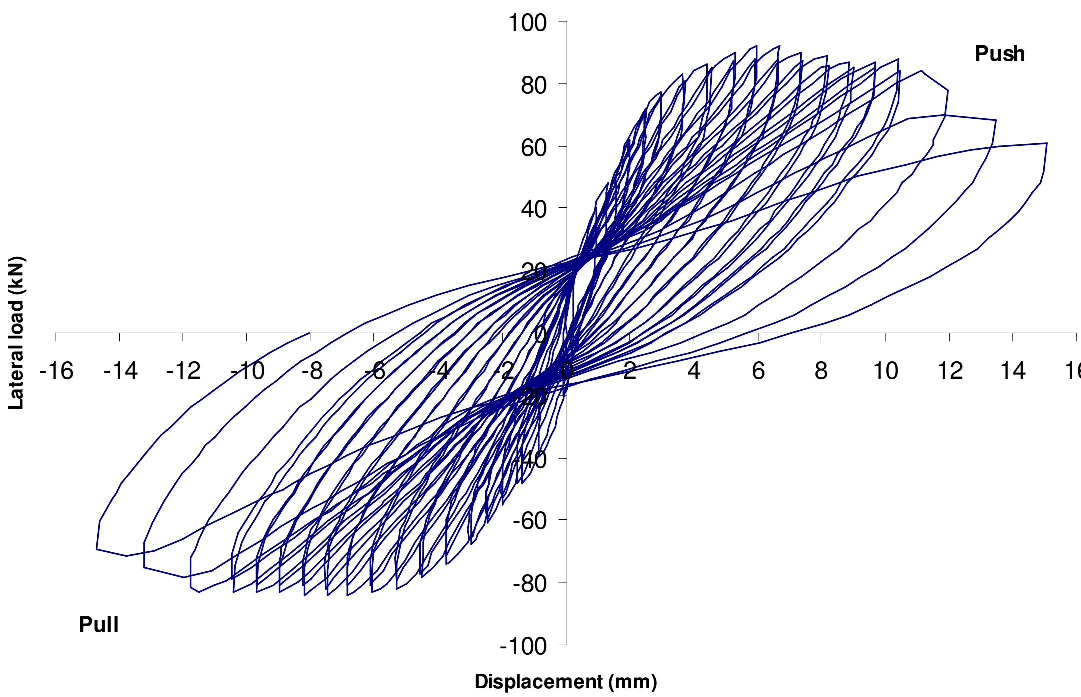

Figure 7. Lateral load-Top displacement relationship of the retrofitted wall RW2.

4 Comparisons of test results

Figure 8 shows the envelope of the lateral load-drift ratio relationships for the three tested walls. The retrofitted wall RW1 showed an increase of the flexural capacity of 80% compared to the control wall accompanied by a decrease of the wall's displacement ductility. Wall RW1 reached a displacement ductility of 5.5 measured at 20% strength degradation after the peak load. The yield load of RW1 was measured to be 46% higher than the control wall at a 7% higher yield displacement. The retrofitted wall RW2 showed an increase of the flexural capacity of 50% compared to the control wall accompanied by similar displacement ductility. The wall reached a displacement ductility of 9.0 measured at 20% strength degradation. The yield load of RW2 was measured to be 19% higher than the control wall. It is worth noting that the retrofitted wall RW2 was able to sustain higher rotation at the wall top compared to the control wall CW, whereas the retrofitted wall RW1 was only able to sustain 65% of the rotation of the control wall. This indicates that the retrofit scheme used for RW2 is able to improve the overall rotational ductility capacity of the wall while increasing its flexural capacity. Therefore, such retrofit scheme will be efficient in the retrofit of multi-storey RC walls at the plastic hinge regions. On the other hand, the retrofit scheme used for wall RW1 is not recommended in case the wall rotational ductility capacity is to be maintained.

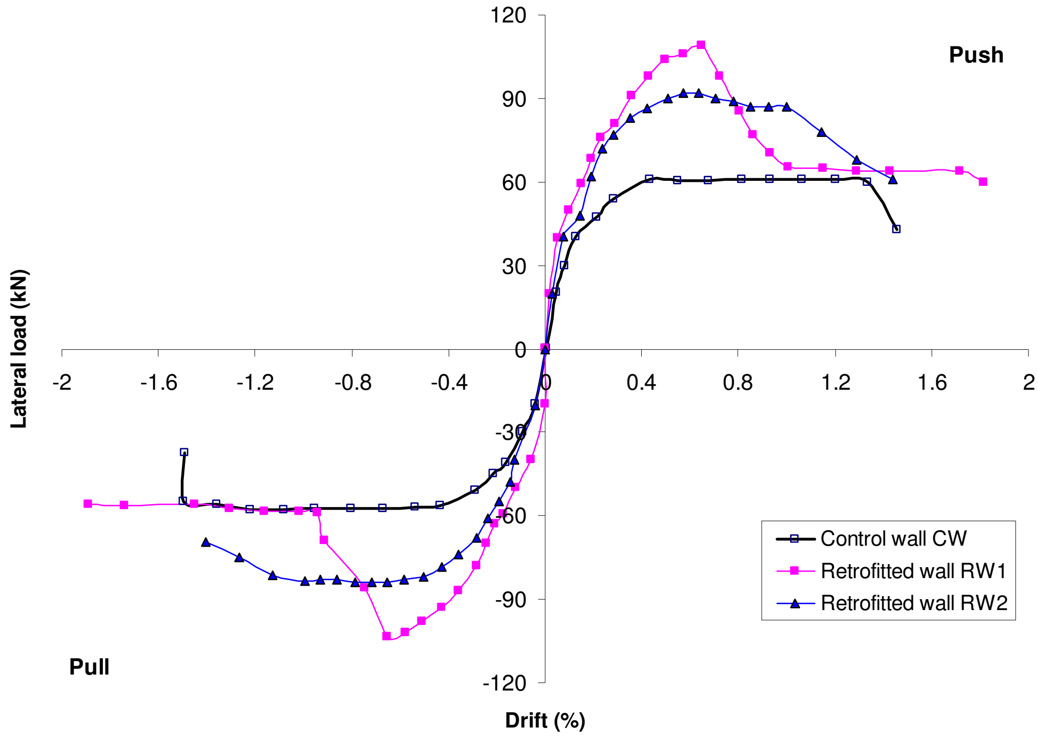

Figure 8. Envelope for lateral load-drift ratio relationships for the tested walls.

5 Conclusions

The seismic behaviour of reinforced concrete (RC) shear walls retrofitted using carbon fibre-reinforced polymers (CFRP) was investigated. The experimental program included testing three RC wall panels under lateral cyclic loading up to failure. The wall panels represent the control wall and two FRP-retrofitted walls using two different retrofit schemes. The main target of the retrofit schemes was to increase the flexural capacity of the wall section as well as its shear capacity to conform to the capacity design philosophy. The FRP-rehabilitated wall panels performed efficiently showing an improved flexural behaviour compared to the control wall. The lateral load capacities at yield for the retrofitted walls RW1 and RW2 were about 46% and 19% higher than that of CW, respectively, occurring at a 7% higher yield displacement. The control wall was able to sustain a displacement ductility of 10.0 measured at an average lateral load of 59 kN. The retrofitted wall RW1 showed an increase of the flexural capacity of 80% compared to the control wall accompanied by a decrease of the wall's displacement ductility. The retrofitted wall RW1 reached displacement ductility, μ_Δ, of 5.5 measured at 20% strength degradation after the peak load. The retrofitted wall RW2 showed an increase of the flexural capacity of 50% compared to the control wall accompanied by similar displacement ductility. The wall reached displacement ductility, μ_Δ, of 9.0 measured at 20% strength degradation after the peak load.

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 du Québécois 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.

References

Antoniades, K., Salonikios, T., and Kappos, A. 2003. "Cyclic Tests on Seismically Damaged Reinforced Concrete Walls Strengthened using Fiber-Reinforced Polymer Reinforcement" *ACI Structural Journal*, 100(4): 510-518.

Canadian Standards Association CSA. 2004. "Design of Concrete Structures A23.3-04", Mississauga, Ont., Canada.

Elnashai, A., Pilakoutas, K., and Ambrayses, N. 1990. "Experimental Behavior of Reinforced Concrete Walls under Earthquake Loading" *Journal of Earthquake Engineering and Structural Dynamics*, 19(3): 387-407.

El-Sokkary, H., Galal, K., Ghorbanirenani, I., Léger, P., Tremblay, R. 2013. "Shake table tests on FRP-rehabilitated RC shear walls" Journal of Composites for Construction (ASCE), 17(1): 1-12.

Fyfe. 2010. "Tyfo® SCH-11UP Composite using Tyfo® S Epoxy User's Guide" *Fyfe Company LLC*, http://www.fyfeco.com.

Ghorbanirenani, I., Tremblay, R., El-Sokkary, H., Galal, K., Léger, P., and Leclerc, M. 2010. "Shake Table Tests and Repair of Ductile Slender Reinforced Concrete Shear Walls" *9th US National and 10th Canadian Conference on Earthquake Engineering*, Toronto, ON.

ISIS Design Manual 4. 2008. "FRP Rehabilitation of Reinforced Concrete Structures". ISIS Canada Corporation, The Canadian Network of Centers of Excellence on Intelligent Sensing for Innovative Structures, Canada.

Kanakubo, T., Aridome, Y., Fujita, N. and Matsui, M. 2000. "Development of Anchorage System for CFRP Sheet in Strengthening of Reinforced Concrete Structures" *12th World Conference on Earthquake Engineering*, (CD-ROM), paper No. 1831.

Khalil, A., and Ghobarah, A. 2005. "Behaviour of Rehabilitated Structural Walls" *Journal of Earthquake Engineering*, 9(3): 371-391.

Lombard, J., Lau, D., Humar, J., Foo, S., and Cheung, M. 2000. "Seismic Strengthening and Repair of Reinforced Concrete Shear Walls" *12th World Conference on Earthquake Engineering*, Auckland, NZ (CD-ROM), Paper No. 2032.

NBCC 2005. "National Building Code of Canada" National Research Council of Canada, Ottawa, ON.

Paterson, J. and Mitchell, D. 2003. "Seismic Retrofit of Shear Walls with Headed Bars and Carbon Fiber Wrap" ASCE *Journal of Structural Engineering*, 129(5); 606-614.

Priestley, M., Seible, F., and Calvi, G. 1996. "Seismic Design and Retrofit of Bridges" Wiley, New York.

Tremblay, R., Léger, P. and Tu, J. 2001. "Inelastic Seismic Response of Concrete Shear Walls Considering P-delta Effects" *Canadian Journal of Civil Engineering*, 28(4): 640-655.