# Cyclic Tests on FRP-Retrofitted RC Shear Wall Panels

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**SUMMARY:**
This paper presents an experimental program that aims to investigate the seismic performance of reinforced concrete (RC) shear walls retrofitted using carbon fibre-reinforced polymer (FRP) composites. Two RC wall panels were tested; one control wall panel and one FRP-strengthened wall panel. 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 wall specimens represent the 6th storey panel of an 8-storey RC wall designed according to the National Building Code of Canada 2005. The main purpose of the FRP-retrofit scheme was to increase the flexural and shear capacities of the tested wall panel and to assess the effectiveness of the FRP-retrofit scheme up to failure. The seismic performance enhancement of the tested wall panels was evaluated.

*Keywords: fibre-reinforced polymers, reinforced concrete, seismic, shear walls, strengthening.*

## 1. INTRODUCTION

In the past few decades, there have been considerable advancements in the design of reinforced concrete (RC) shear walls for new construction such as performance based seismic design and capacity design principles. This resulted in a concurrent need for upgrading the seismic performance of existing RC shear walls such that they can meet safety requirements of modern seismic design codes. As such, there would be a need to retrofit existing RC structural 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).

Several retrofit techniques of RC shear walls using different materials were reported in the literature. These ranged from using steel, concrete, fibre-reinforced polymer composites, 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. The use of fibre-reinforced polymer (FRP) composite materials has 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 use of FRP composites offers also a faster and easier retrofit alternative, especially when the evacuation of the entire building during the retrofit is not possible. In that case, FRP would provide the required strength and/or ductility without interrupting the use of the building.

The wall flexural capacity can be enhanced by adding FRP at the extremities of the cross section and orient the fibres parallel to the wall axis. FRP sheets would be 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. Paterson and Mitchell (2003) retrofitted a RC shear wall using CFRP wraps and through-thickness headed reinforcement in order to increase the wall shear strength and confinement. The retrofitted wall was able to reach a displacement ductility 57% higher than the control wall, and it was able to dissipate three times the energy absorbed by the original wall. Khalil and Ghobarah (2005) tested two RC wall panels rehabilitated using FRP composites to increase the shear capacity and ductility of the walls. They found that the lateral load capacity has increased by about 40 and 57% for the first and second wall, respectively. The two rehabilitated walls were able to reach displacement ductilities of 3.0 and 4.0 at their maximum strength compared to displacement ductility of less than 1.0 for the control wall. Both flexural and shear capacities can be also enhanced by applying the fibres in both directions or by using diagonal strips (Lombard et al. 2000).

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. Two RC shear wall panels will be tested under cyclic loading up to failure. The tested walls represent the control wall and the FRP-retrofitted one.

## 2. EXPERIMENTAL PROGRAM

### 2.1. Test specimens and setup

Two wall panels were constructed and tested. The test wall panel represent the 6$^{th}$ 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 FRP-retrofitted one RW1 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. In order to meet the similitude between the model and prototype, the steel bars were unbonded in order not to contribute to the flexural resistance of the wall panel. The wall dimensions and reinforcement are shown in Figure 1.

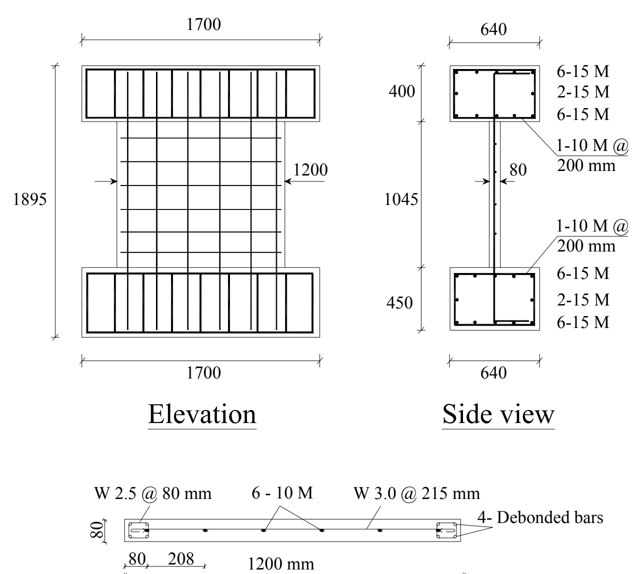


**Figure 1.** The wall panel specimen and its reinforcement.

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 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 control wall panel and the FRP-retrofitted ones 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 loading I-beam was used to transfer the actuator forces to the wall top block uniformly. Two steel double angle braces were connecting the rigid I-beam to the laboratory wall. The steel braces were designed to guide the steel loading beam and allow a smooth in-plane movement of the wall panel. The steel braces 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 the failure phase.

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:

$$F_A = 24 + 1.115\, F_C \text{ (kN)} \quad (2.1)$$
$$F_B = 24 - 1.115\, F_C \text{ (kN)} \quad (2.2)$$

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.

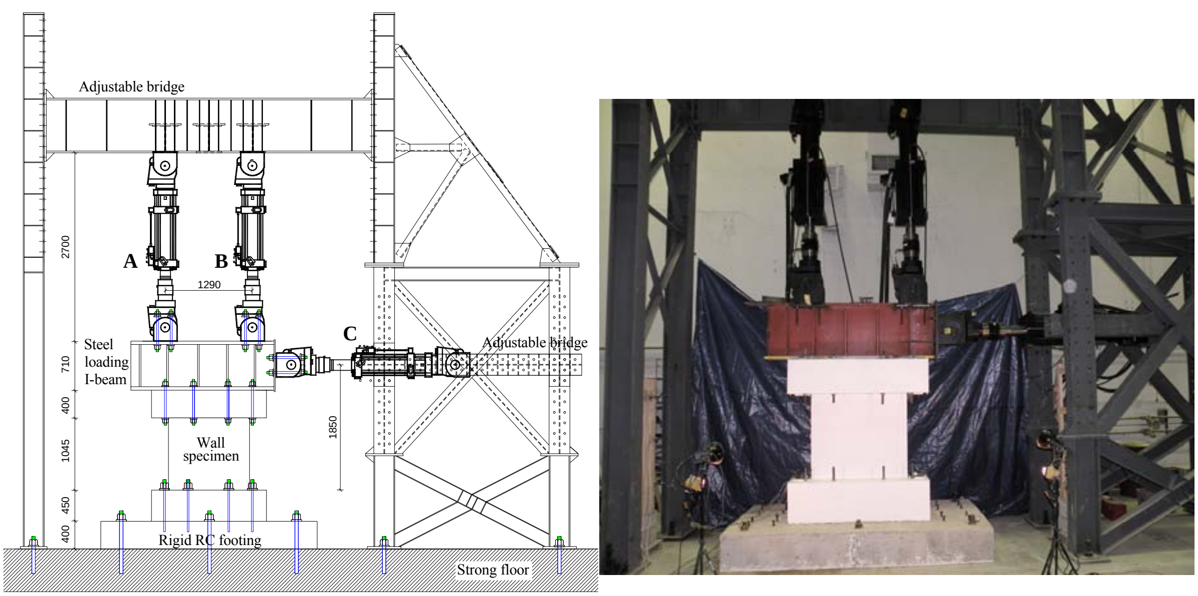


**Figure 2**. Test setup of the two wall panels.

Carbon fibre-reinforced polymer (CFRP) composites were used for the retrofit of the wall panel. Tyfo® SCH-11UP composite system (Fyfe 2010) with uni-directional CFRP sheets was used for the retrofitted wall. 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. A total of 16 anchors were used for the retrofitted wall specimen. 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-rehabilitation (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 panel CW was tested under static cyclic loading up to failure. The control wall panel represents the 6$^{th}$ 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 the strain hardening of steel reinforcement into account. 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. Retrofit scheme for RW1

The main target of both retrofit schemes was to enhance the seismic performance of the tested wall panels by increasing the flexural capacity of 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 (Ghorbanirenani et al. 2010). From the shake table tests conducted on the 8-storey walls, it was found that the factored moment at the 6$^{th}$ 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 wall 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 wall RW1. As a result of increasing the wall's flexural capacity, the retrofit scheme 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 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). A material resistance factor $\varphi_{FRP}$ 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 RW1 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. The retrofitted wall RW1 prior to testing is shown in Figure 4.

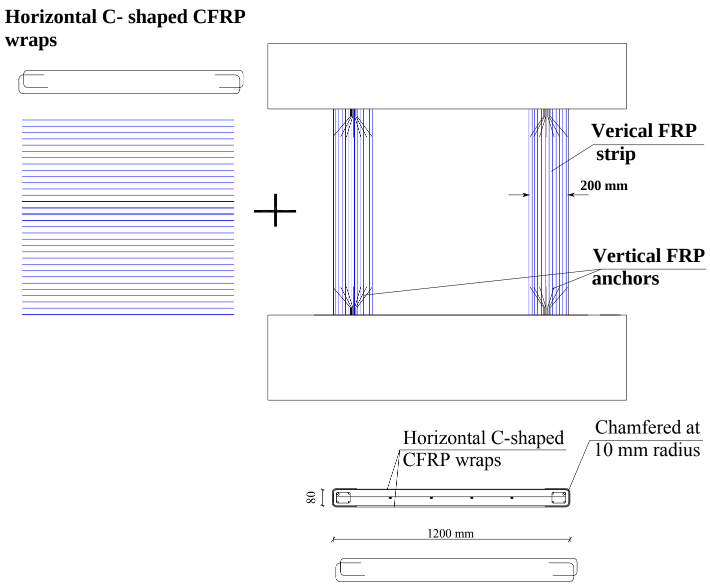


**Figure 3.** FRP retrofit scheme of RW1.

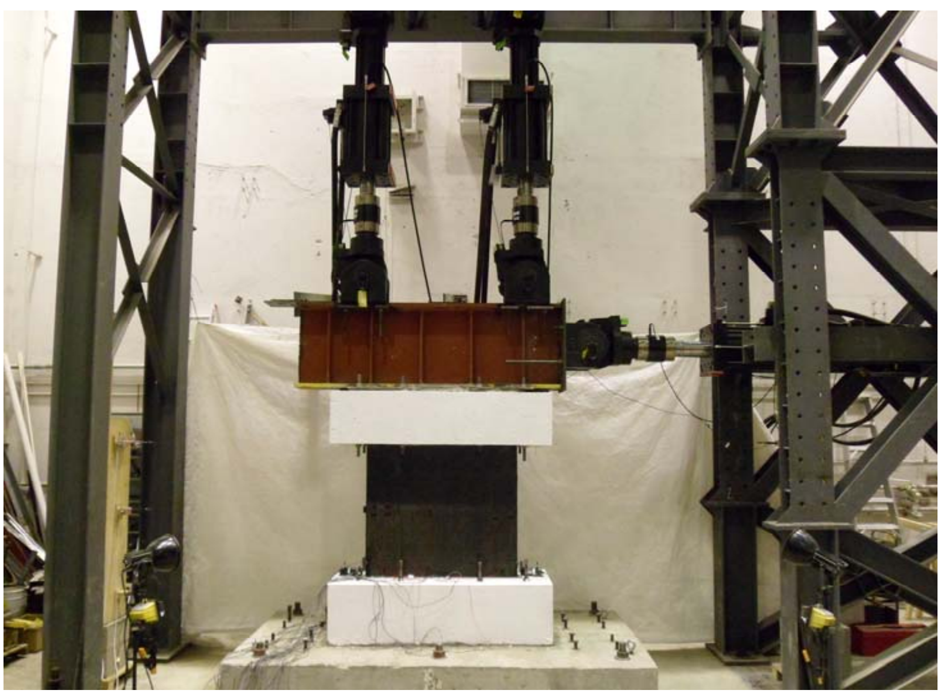

**Figure 4.** FRP-retrofitted wall RW1 prior to testing

## 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 ($\mu_\Delta = 3.0$) 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 ($\mu_\Delta = 3.0$). 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 $\mu_\Delta = 8.0$ and a drift ratio of 1.08%.

The control specimen was able to sustain a lateral displacement of 14 mm, which corresponds to $\mu_\Delta =$ 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 ($\mu_\Delta = 11.0$) 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 $\mu_\Delta = 10.0$. The failure mechanism of the control wall was rupture of the extreme flexure reinforcement bars accompanied by concrete crushing of the wall toes.

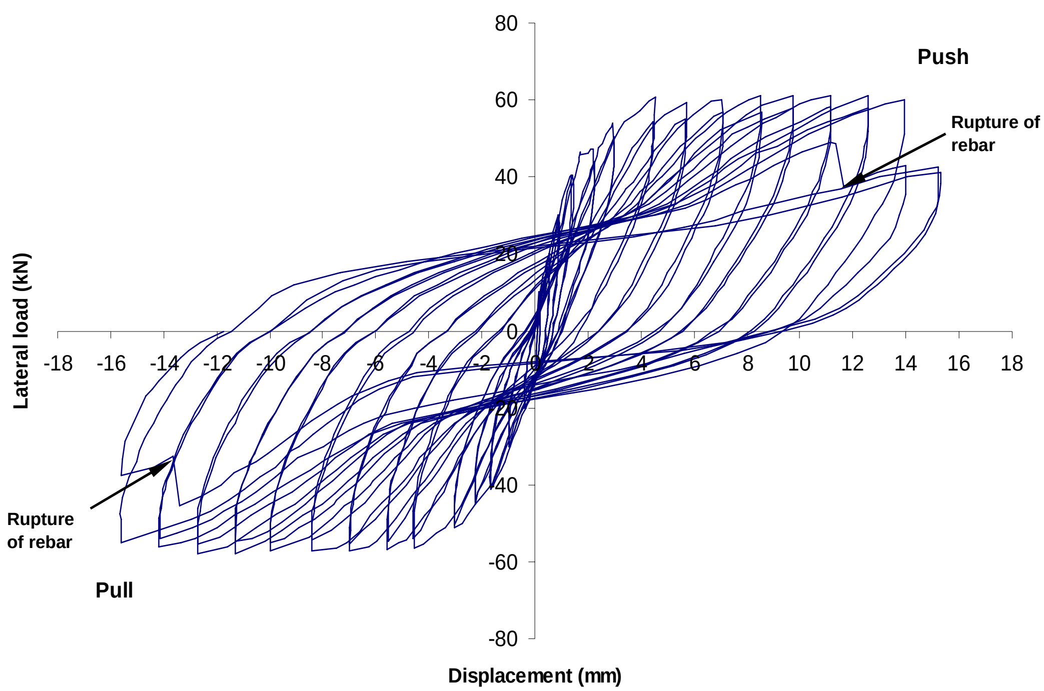


**Figure 5.** Lateral load-Top displacement relationship of the control wall CW.

### 3.2. 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 $\mu_\Delta = 4.5$ 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 ($\mu_\Delta = 5.0$), 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 ($\mu_\Delta = 6.0$), the strength of the retrofitted wall RW1 reached almost that of the control wall in the pull direction. At a lateral displacement of 10.5 mm ($\mu_\Delta = 7.0$), 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.

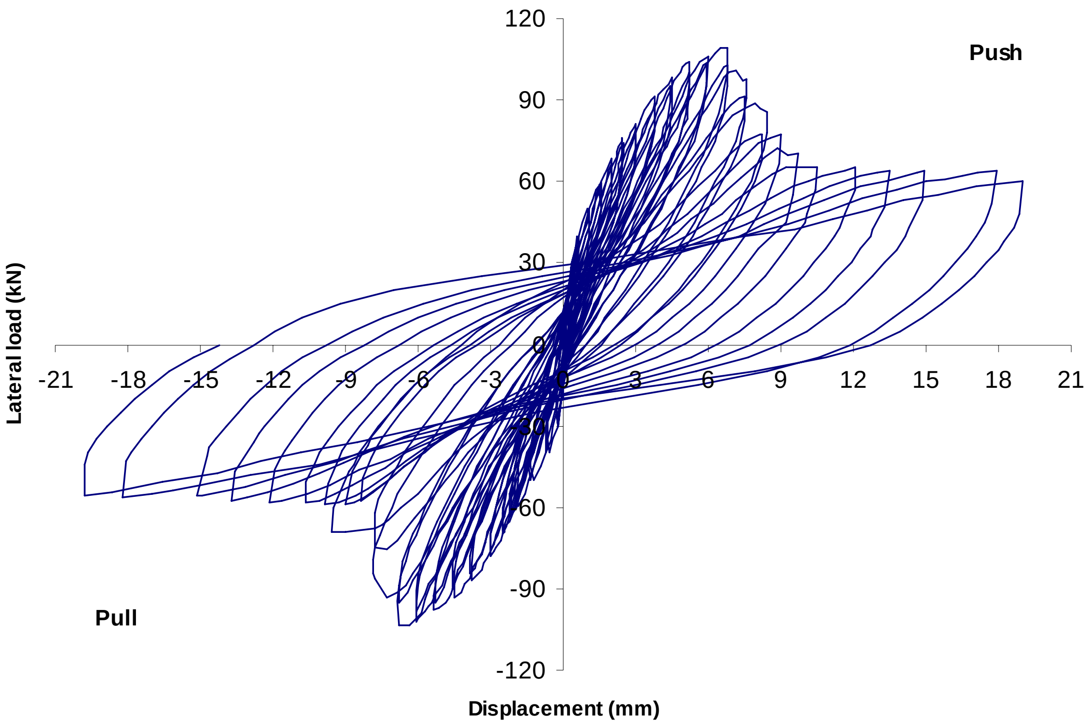


**Figure 6.** Lateral load-Top displacement relationship of the retrofitted wall RW1.

## 4. COMPARISONS OF TEST RESULTS

Figure 7 shows the envelope of the lateral load-drift ratio relationships for the two 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 RW1 was only able to sustain 65% of the rotation of the control wall. Therefore, such retrofit scheme is not recommended in case the wall rotational ductility capacity is to be maintained.

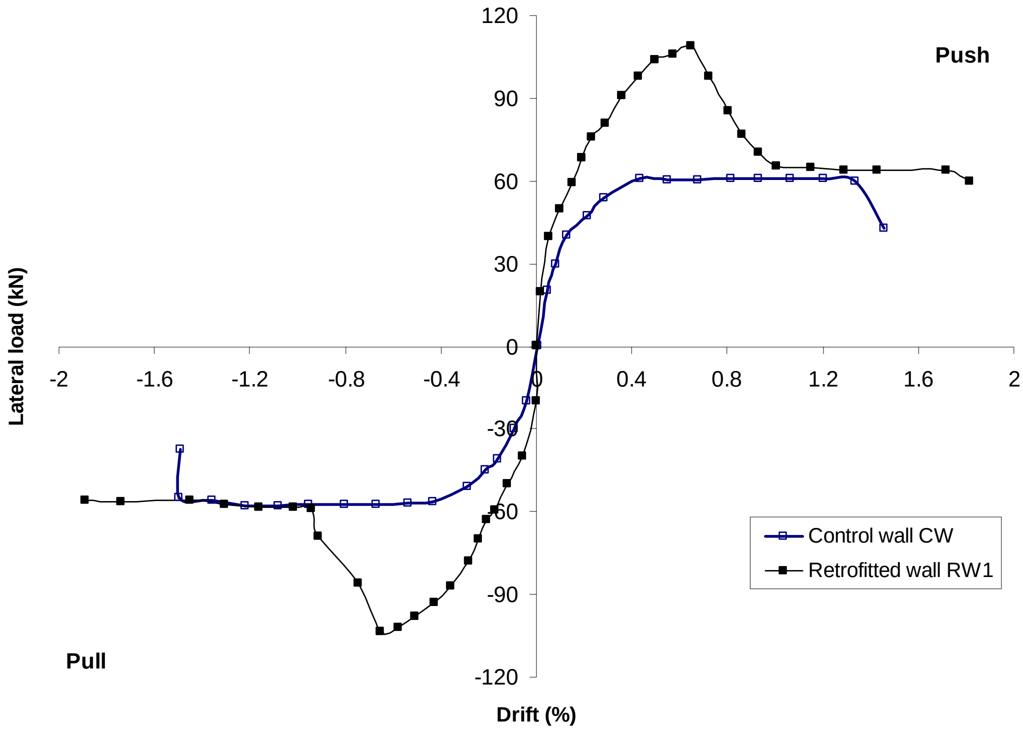


**Figure 7.** Envelope for lateral load-drift ratio relationships for the tested walls.

## 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 two RC wall panels under lateral cyclic loading up to failure. The wall panels represent the control wall and the FRP-retrofitted one. The main target of the retrofit scheme 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 panel performed efficiently showing an improved flexural behaviour compared to the control wall. 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 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, $\mu_\Delta$, of 5.5 measured at 20% strength degradation after the peak load.

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