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Seismic Retrofit of R.C. Shear Walls with Externally Bonded FRP Tow-Sheets

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Abstract: This paper presents a study on the seismic repair and strengthening of deficient reinforced concrete shear walls with externally-bonded FRP tow sheets. The investigation includes analytical studies and the design of a comprehensive experimental program. The objective of the study is to develop FRP reinforcing schemes to enhance the seismic behaviour of shear wall buildings designed according to old code provisions. These structures exhibit a number of structural problems such as insufficient shear reinforcement, lap splices at the plastic hinge region, and poor concrete confinement at the boundaries. The pre-experimental section of the study consists of the cyclic testing up to failure of nine shear wall specimens with aspect ratios ranging from 0.65 to 1.20. The wall specimens have insufficient shear reinforcement, poor confinement, and two specimens include lap splices located at the plastic hinge region. The use of FRP is investigated in both repair and strengthening applications. The FRP material is used in both the vertical and horizontal direction of the walls to enhance the stiffness, flexural strength, shear capacity, ductility and concrete confinement of the walls. Preliminary analytical results show that the FRP material is effective in eliminating the brittle shear failure mode in walls with insufficient shear reinforcement.

1 Introduction

Reinforced concrete (RC) shear walls are a widely used lateral load resisting system commonly present in structures located in seismically active zones. Shear walls have a higher amount of lateral stiffness in the in plane direction than other structural elements and are expected to perform better in resisting lateral loads. Although significant advancements in seismic design have been made in recent decades (ACI318-05, CSA A23.3 04), older shear wall buildings are still at risk of suffering considerable damage during moderate or large earthquakes. This is attributed to insufficient in plane stiffness, insufficient flexural and shear capacities, insufficient concrete confinement and poor detailing (Lombard et al. 2000). Various repair and retrofit schemes are available to mitigate the low seismic response resulting from walls with structural deficiencies. Some of the repair and retrofit techniques deal with the strengthening of the existing walls by adding additional walls or bracings to increase the stiffness of the structural elements. Although these techniques are effective in improving the seismic response of structures, they are labour intensive and can be quite disruptive to the occupants and the functioning of the facility. They may also alter the distribution of lateral loads on the building by adding more weight to the structure (Lombard et al. 2000). Another effective, minimally disruptive option for repair and strengthening is the use of carbon fibre reinforced polymers (CFRP). In recent years, fibre reinforced polymers (FRP) have become commonly used structural materials due to factors such as ease of application, high strength to weight ratio, and high resistance to corrosion (Meier et al. 1992). Previous research conducted on shear wall elements retrofitted with FRP was focused on addressing the increase in shear strength and energy dissipation (Antoniades et al. 2003; Paterson and Mitchell 2003; Khalil and Ghobarah 2005). However, few studies (Lombard et al. 2000; Hiotakis 2004) have investigated the increase in flexural strength. In this study, the efficiency of externally-bonded FRP sheets in repair and strengthening applications of shear wall specimens detailed according to older code provisions (ACI 1968, CSA 1977) will be evaluated. Finite-element studies are conducted to determine the ultimate strength and failure modes of the wall specimens. The increases in stiffness, strength, and ultimate displacement capacity are discussed.

2 Experimental Setup & Design Methodology

2.1 Scope of Experimental Program and Design Procedures

The use of externally bonded CFRP sheets in repair/strengthening applications for RC shear walls with structural deficiencies typical of older construction codes is to be investigated in a comprehensive study at Carleton University. Nine large-scale shear wall specimens are to be subjected to quasi-static cyclic loading up to failure. The structural deficiencies in the walls are intended to represent typical design details in older construction codes (ACI 1968, CSA 1977), and include poor confinement, non-seismic detailing, and non-ductile details (lap splices) at the plastic hinge region. The nine shear wall specimens have different aspect ratios, *h/l,* where *h* is the height of the wall and *l* is its length. The experimental setup includes testing of two wall specimens with *h/l=1.2, three walls with h/l=0.85, and four walls with h/l=0.65 (Table1).* Two of the four walls with *h/l=0.65 have lap splices at the plastic hinge region*. All wall specimens have 3% longitudinal reinforcement ratio and 0.25% transverse reinforcement ratio in their unrepaired and un-strengthened state, which ensures that the walls will exhibit a brittle shear response before their flexural capacity is reached. The externally-bonded CFRP sheets will be applied in both the vertical and horizontal directions. The retrofitting scheme aims to facilitate a ductile flexural behaviour in the wall specimens, preventing any premature shear failure from occurring. Since the walls are poorly confined, the FRP material will be used to improve the confinement of the concrete by wrapping the horizontal sheets around the wall. The FRP system will be investigated in both repair and strengthening applications, with some specimens with no FRP reinforcement being tested as a reference (control walls). After the testing of the control wall specimens, the wall specimens will be repaired and retrofitted with FRP reinforcement as shown in table1. The loading system is comprised of an ENERPAC actuator, two steel hinges and one steel reaction frame. An additional frame will be used to prevent out of plane deformations of the wall specimens. The shear wall flexural and shear capacities were determined according to current code provisions (ACI318-05, CSA A23.24 2004).

Table1: Repair/Strengthening Scheme

Wall Type	Aspect Ratio (as)	Vertical sheets*	Horizontal sheets*	Total sheets per side
Slender wall	1.2	1V	3H	1V+3H
Slender wall	1.2	1V	3H	1V+3H
Squat wall-1	0.85	1V	3H	1V+3H
Squat wall-1	0.85	1V	3H	1V+3H
Squat wall-1	0.85	1V	3H	1V+3H
Squat wall-2	0.65	-	4H	4H
Squat wall-2	0.65	-	4H	4H
Squat wall-2	0.65	3V	4H	3V+4H
Squat wall-2	0.65	3V	4H	3V+4H

*nV= wall reinforced with n number of layers in the vertical direction per one side of the wall specimen.
*nH= wall reinforced with n number of layers in the horizontal direction per one side of the wall specimen.

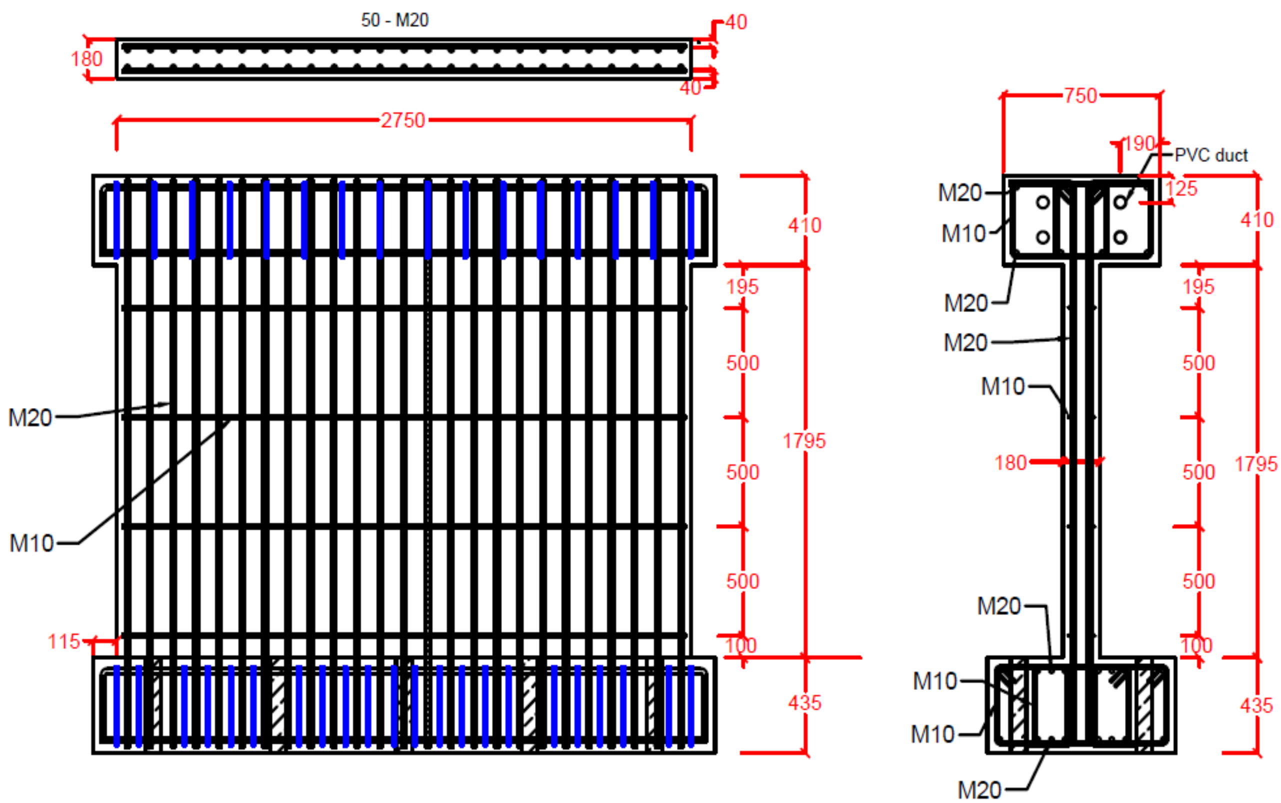

Figure 1: Squat wall-2 Specimen Detailing (all dimensions are in mm). Cruz-Noguez et al. (2012), 6th International Conference on Advanced Composite Materials in Bridges and Structures, ACMBS-VI, Kingston, Ontario, Canada, 2012.

2.2 Strength Calculations

This section illustrates the procedures used to determine the flexural and shear strength of the walls with no FRP reinforcement, and the strength against sliding shear failure. The sliding shear strength was computed for the wall specimens according to CSA A23.3-04 clause (11.5) and it was found that the sliding shear capacity exceeded the flexural capacities of the walls, which were calculated through conventional section analyses. To calculate the strength against diagonal tension failure (V_r) in the plain R.C. walls with no FRP reinforcement, three different approaches were utilized. The first approach was the semi-empirical equation for seismic design of shear walls given by the ACI318-05 code specification clause (21.7.4.1(21-7)), equation [1], which is based on the modified truss analogy approach:

$$[1]\ V_{r1} = \left(\alpha_c \sqrt{f_c'} + \rho_h f_{yh}\right) A_w \leq 0.83\sqrt{f_c'} A_w$$

Where the term $0.83\sqrt{f'_c}$ (MPa) is a limit specified by clause (21.7.4.5) intended to prevent diagonal compression failure. α_c in the equation is an aspect ratio coefficient taken as 0.25 for $h_w/l_w<1.5$. ρ_h is the horizontal web reinforcement ratio, f_{yh} is the yield stress of the horizontal web reinforcement. f'_c is the compressive strength of the concrete and A_w is the area of the wall. The second method was also based on the modified truss analogy approach and is a semi-empirical equation for the general design with special provisions for walls detailed by the ACI318-05 code specification section (11.10). In this approach the concrete contribution is taken into account using normal shear strength calculation specifications. The resulting diagonal tension shear is the summation of both the contributions provided by both steel and concrete. The contribution from the concrete is taken as the lesser of equations [2] & [3] presented below:

[2] $V_{C1} = 0.27\left(\sqrt{f'c}\right)hd + \frac{Nud}{4lw}$

Where h is the thickness of the wall specimens, d is the distance which is set by the limits of the equation specified by clause (11.10.4) as 0.8 lw , where lw is the overall length of the wall. N_u in the equation is negative if in compression and positive if in tension.

[3] $V_{C2} = 0.05\left(\sqrt{f'c}\right) + \frac{lw\left(0.1\sqrt{f'c} + 0.2\frac{Nu}{lwh}\right)}{\frac{Mu}{Vu} - \frac{lw}{2}}$

Where the term $M_u/V_u-l_W/2$ must not be negative in equation [3], set as a limitation by clause (11.10.6). If the term is negative then equation [3] must not be used and the concrete contribution will be solely from equation [2]. The contribution from steel is computed by the use of equation [4] shown below specified by clause (11.10.9.1):

[4] $Vs = \frac{Avfyhd}{S}$

Where A_V is the area of horizontal shear reinforcement within a spacing S, and d is the distance determined as specified by clause (11.10.4). Diagonal shear is then computed by summing up the contributions obtained from both the concrete and steel as shown below in equation [5].

[5] $Vr2 = \min(V_{C1}, V_{C2}) + Vs$

The third approach in calculating diagonal shear was utilizing Wiradinata's Method (1985) which takes into account the increase in shear capacity due to a reduction in the aspect ratio of the wall. In this method diagonal tension shear is obtained from the contributions of both the diagonal concrete struts and the transverse shear reinforcement. The contribution of the diagonal concrete struts is first presented by equation [6] and then the contribution of the steel is presented in equation [7], the summation of both contributions should be less than the limit specified in equation [1].

[6] $Vc = \left(0.5 - \frac{h}{6w}\right)Ae\sqrt{f'c}$

A_e is the effective area of the wall (considered to be 90% of the cross-sectional wall area), and f'_c is the compressive strength of concrete after 28 days. While the transverse steel contribution is given by equation [7]:

[7] $V_S = \left(\frac{nA_h f_{yh} D'}{S}\right)\cot\theta$

A_h in equation [7] is the area of one leg of transverse reinforcement. N is the number of reinforcing bars per layer, S is the spacing between the transverse reinforcing bars, θ is the angle of the critical inclined flexural/shear crack with respect to the longitudinal axis of the wall, f_{yh} is the yield stress of the horizontal steel reinforcement. D' is the effective depth of the wall which is taken to be ≥ $0.8l_{W}$. As suggested by Wiradinata θ is taken to be equal to $45°$.The end result of diagonal shear is presented in equation [8]:

$$[8]\ V_{r3} = V_c + V_s \leq 0.83\sqrt{f'_c}A_w \leq V_{sl}$$

V_{sl} in equation [8] represents sliding shear, which is a limit imposed on Equation [1] where the value of sliding shear has to exceed the upper limit of diagonal shear set by ACI318-05 clause (21.7.4.1). Sliding shear was calculated by the use of CSA A23.3 (clause 11.5). It was assumed that the crack occurs along the shear plane of the element. The relative displacement is assumed to be resisted by cohesion and friction in the shear friction reinforcement crossing the crack. The factored shear stress resistance of the plane is computed according to equations [9] & [10]:

$$[9]\ Vsl = [c + \mu(\frac{Asfy}{hd} + \frac{Nu}{hlw})]Aw$$

Where V_{sl} is the sliding shear parameter, c & µ are cohesion and friction factors specified by CSA A23.3 clause (11.5). A_S is the area of vertical reinforcement bars, f_y is the yield stress associated with vertical reinforcement. An upper limit shear strength computation is provided in equation [10] and the lesser of the two values is taken to be the expected sliding shear strength.

$$[10]\ Vsl = 0.25 f'cAw$$

A comparison between the different diagonal shear determination methods, flexure and sliding shear is shown in figure 2.

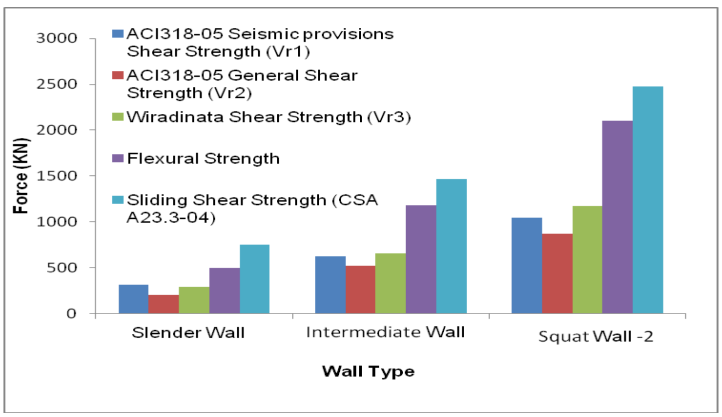

Figure 2: Comparison of Shear & Flexure Computations.

3 Pre-Test Analytical Studies (Modelling Approach & Specimen Models)

This section discusses the pre-test analytical studies conducted on nine shear wall specimens using the finite element method. Analytical modelling of the wall specimens was carried out using a robust finite element program, Vector 2 (Wong and Vecchio, 2002). The analysis started with developing models for regular concrete walls without the addition of externally bonded FRP sheets. The models were generated by determining distinct concrete regions which represented the various parts of the wall specimens (cap beam, foundation block, and wall element).

3.1 Material Properties

The concrete material properties used within the concrete region include a compressive strength f'_c=20 Mpa, tensile strength of concrete f'_t =0 Mpa. The steel reinforcement within the concrete region has a yielding stress of f_y=412 Mpa and an associated yielding strain ε_y= 0.00206. The ultimate strength of the steel used was f_u=654 Mpa and the associated ultimate strain ε_u=0.1126. The onset of the strain hardening was ε_{sh}=0.0126 and the strain hardening modulus E_{sh}=2420 Mpa. The properties of the dry carbon fibre material are a tensile strength f_{FRPu} of 3480 MPa, yield strength f_{yFRP}=1740 Mpa, tensile modulus E_{FRP} of 230 GPa, and thickness t_{FRP} of 0.11 mm.

3.2 Material Formulations

The material modelling and formulation included assigning the concrete material as an orthotropic material with smeared rotating cracks (Wong and Vecchio, 2003). The steel reinforcement material was modelled as an elasto-plastic material with strain hardening effects. The CFRP material was modelled as a brittle material with zero compressive strength, with tensile capacity only. The bond stress-slip relationship used was that of a brittle trilinear behaviour (Cruz-Noguez et al. 2012). The smeared crack approach was used to model the cracking of the concrete. In the smeared crack approach an infinite number of parallel cracks with infinitely small width are considered to be evenly distributed or “smeared” over the element. This approach was implemented in the study due to its simplicity while in the same time being accurate (Cortés Puentes 2009; Cruz-Noguez et al. 2012).

3.3 Element Formulations

Four node quadrilateral elements are used to model the concrete regions as shown in figure 3. The steel reinforcement was smeared within the concrete region. The FRP material was modelled using discrete truss elements. The interaction between the CFRP sheets and the concrete region was represented by zero length link elements. Link elements are elements which create contact between two materials through defining bond stress-slip relationships.

3.4 Material Models & Wall Geometry

The material models used in the analysis are the Popovics concrete model for the pre-peak response, and the Popovics/Mander model for the post-peak response. The geometry of the wall specimens consists of three concrete regions. The concrete regions include material properties of both concrete and steel reinforcement. The concrete regions include various parts of the wall specimens (cap beam, foundation block, and wall element). The cap-beam and the foundation block were modelled as rigid elements. Connection between the FRP and the concrete at the base of the wall follows the common node approach since mechanical anchorage is provided at the base. Fixed supports at the base of the wall were imposed due to the foundation being held by diagonal anchors to the laboratories strong floor.

3.5 Debonding Mechanism

During previous tests on shear wall specimens flexurally-reinforced with externally-bonded CFRP sheets (Lombard et al. 2000; Hiotakis 2004), significant debonding between the FRP and the concrete substrate was observed. Debonding in flexural elements occurs when the interfacial stresses between the FRP material and the concrete exceed the bond stresses generated between the two materials. The debonding mechanism observed in the tests conducted by Hiotakis (2004) and Lombard (1999) was first initiated by the opening up of major flexural cracks within the concrete. This type of debonding is known as intermediate crack debonding (also called IC debonding). According to Cruz-Noguez et al (2012), the crack width that leads to debonding between FRP and concrete was determined as 0.058 mm for concrete with compressive strength of 40 MPa. The debonding mechanism is modeled in this study using a dual debonding criterion which monitors the crack widths of the concrete elements as suggested by Cruz Noguez et al (2012). If the crack width exceeds a certain critical slip value, debonding of the FRP material occurs.

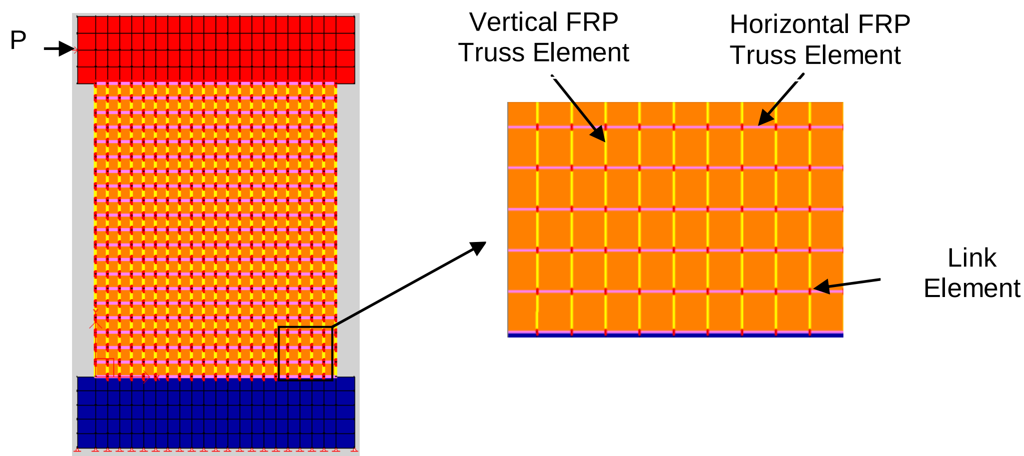

Figure 3: Finite Element Model of Slender FRP Wall Specimen & Components.

4 Finite Element Analysis Results & Discussion of Wall Specimens

The wall specimens were subjected to a displacement controlled reverse cyclic loading up to failure. A comparison was conducted between the plain concrete wall specimens and their FRP strengthened counterparts (figures 4, 5, 6). Energy dissipation models have also been generated for both types of models (without & with FRP) and the representative energy dissipation models showed the cumulative energy being dissipated per load step (figure 7). The hysteresis curves are presented below for each type of wall.

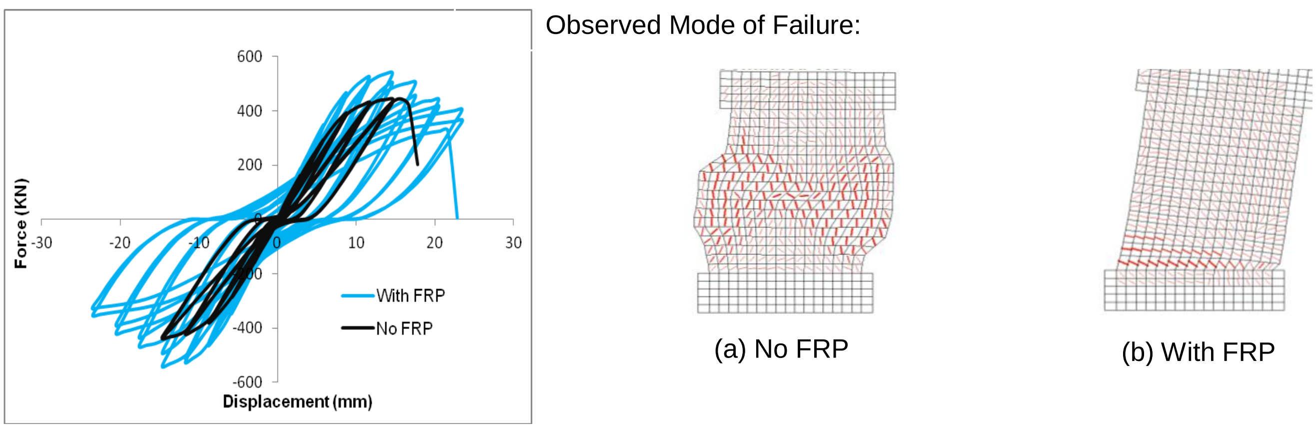

Figure 4: Hysteresis Curve for slender wall (with & without FRP).

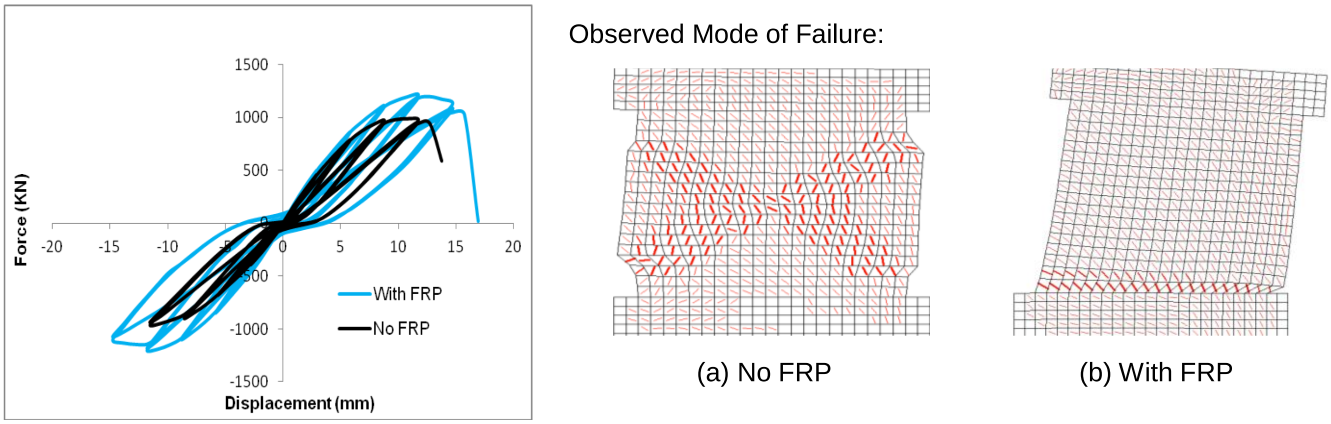

Figure 5: Hysteresis Curve for Intermediate wall (with & without FRP)

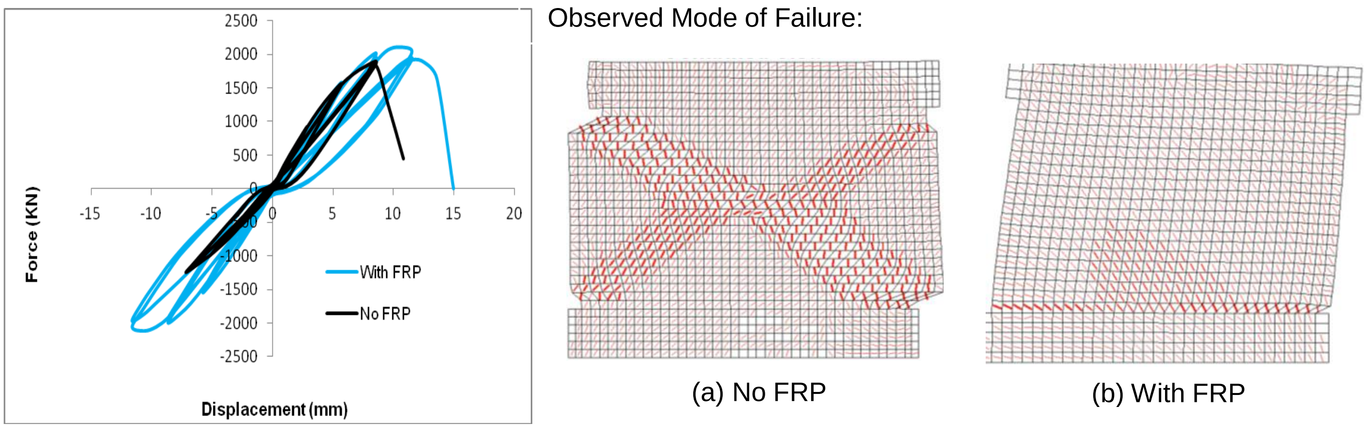

Figure 6: Hysteresis Curve for squat wall-2 without lap splice (with & without FRP).

As shown within the hysteresis curves presented above, the contribution of FRP is evident in enhancing the stiffness, ductility, and strength of the wall specimens. Energy dissipation plots were generated to reflect the accumulated amount of energy dissipation per load step through the analysis.

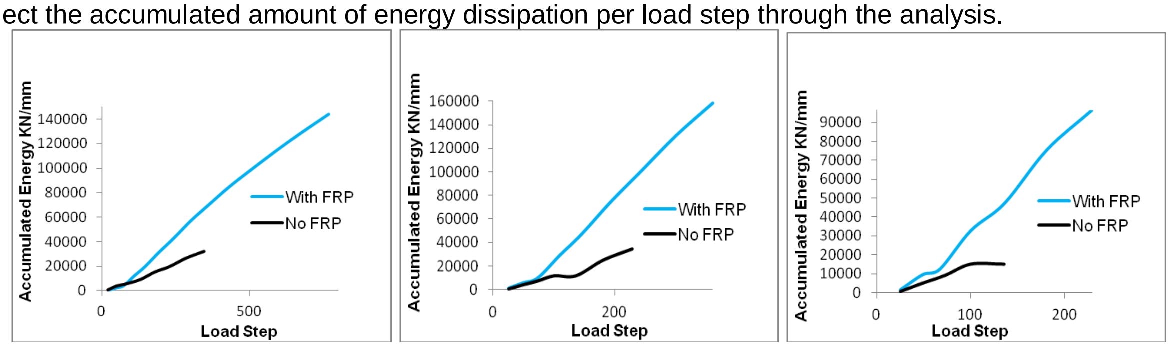

Figure 7: Energy Dissipation Curves for (slender, Intermediate & squat wall-2) with & without FRP.

As presented in the plots above, the FRP reinforcing scheme adds a considerable increase in the energy dissipation capability of each wall specimen. The results concluded from the analysis of each wall specimen with and without FRP are included withinin tables 2, 3.

Table 2: Finite Element Analysis Results

Wall type:	Yielding Stiffness, Ky No FRP (KN/mm)	Yielding Stiffness, Ky With FRP (KN/mm)	Ultimate Strength, Pu No FRP (KN)	Ultimate Strength, Pu With FRP (KN)	Ultimate displacement, Δu No FRP (mm)	Ultimate displacement, Δu With FRP (mm)	Energy Dissipation No FRP (KN/mm)	Energy Dissipation With FRP (KN/mm)
Slender wall	55.791	63.946	444.5	542.8	17.805	23.582	31943.52	144411.6
Intermediate wall	155.786	169.675	987.4	1212.1	13.775	16.943	34325.87	158755.5
Squat wall-2 without lap splice	306.635	317.385	1884.1	2113.8	10.806	14.954	14895.16	96611.78

Table 3: Comparison between FRP reinforced / unreinforced specimens

Wall Type:	% increase in Ky	% increase in Pu	% increase in Δu	% increase in Energy Dissipation Capacity
Slender wall	14.6%	22.11%	32.45%	325%
Intermediate wall	9.0%	22.75%	23%	362%
Squat wall-2 without lap splice	3.5%	12.19%	38%	548.6%

Table 3 shows that the FRP reinforced walls had in average 9% higher stiffness, 19% higher strength, 31.15% higher ultimate displacement, and a 412% higher energy dissipation capability than walls with no FRP reinforcement. Previous studies (Lombard et al. 2000; Hiotakis 2004) reported increases in stiffness (28%) and (40%) for slender walls with the same amount of vertical FRP reinforcement. However, in those studies the steel longitudinal reinforcement ratio was 0.8%, while the longitudinal ratio used in this study was 3%. Thus, the contribution of the FRP material was more significant in the walls tested by Lombard (1999) and Hiotakis (2004). Table 3 clearly shows the benefits of an FRP reinforcing system consisting of vertical and horizontal FRP sheets used simultaneously. Premature shear failure was prevented by enhancing the shear strength of the wall specimens (with the FRP sheets applied in the horizontal direction), allowing the wall specimens to reach their ultimate flexural capacities. The flexural strength and yielding stiffness were also increased by the use of vertical sheets of FRP material. In terms of energy dissipation the FRP reinforced walls exhibited an average of 412% increase which can be translated to a significantly better seismic performance of FRP-reinforced wall specimens.

5 Conclusion

This paper presented the pretest analysis and experimental setup for the seismic testing and analytical modelling of nine R.C. shear wall specimens designed according to older construction specifications (ACI 1968, CSA 1977). The system was shown to enhance the seismic response of the walls in terms of stiffness, energy dissipation, ductility, and strength. The repair and retrofit scheme consists of FRP sheets oriented in the vertical and horizontal direction. The system was shown to eliminate the brittle shear failure associated with shear. The conclusions that can be drawn from this study are the following:

- Pre-test finite element simulations show that the FRP system prevents brittle shear failure in the walls under study and enhances the seismic response thus facilitating a more ductile flexural behaviour.
- FRP system was shown to increase the strength, stiffness, and ultimate displacements of the wall specimens.
- Force- displacement relationships show increased energy dissipation capability in walls with FRP reinforcement.

6 Acknowledgment

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