

Shake Table Tests on FRP-Rehabilitated RC Shear Walls

H. El-Sokkary¹; K. Galal, M.ASCE²; I. Ghorbanirenani³; P. Léger, M.ASCE⁴; and R. Tremblay, M.ASCE⁵

Abstract: This paper investigates the behavior of two 8-story cantilevered RC shear walls rehabilitated using carbon fiber-reinforced polymer (CFRP) composite sheets when subjected to base excitations from a shake table. The two original reduced size walls (1:0.429) were tested on the shake table of École Polytechnique de Montréal to investigate the effect of higher modes of vibration on the behavior of multistory RC walls. The walls were subjected to several levels of ground motion excitation that matches the design spectrum of Montréal city in Québec, Canada. The original walls showed significant inelastic deformations at the 6th-story level in addition to those at the base plastic hinge. After the shake table tests on each of the two original walls, the damaged walls were rehabilitated and resubjected to the same levels of the ground motion excitations. This paper focuses on the FRP-rehabilitation of the original walls. The rehabilitation scheme for the two walls aimed to increase the flexural and shear capacities of the wall at the 6th-story panel because of the observed increase in demand at that level, whereas the base panel was confined using CFRP sheets to increase the ductility capacity without increasing strength. The rehabilitated walls showed satisfactory performance with improved flexural strength at the 6th-story panel. The rehabilitation scheme resulted in a reduced wall rotation and lower strain values of the flexural steel rebars at the 6th-story panel. The shear demands and bending moments on the FRP-rehabilitated walls were higher than those of the original ones. DOI: 10.1061/(ASCE)CC.1943-5614.0000312. © 2013 American Society of Civil Engineers.

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

Author keywords: Reinforced concrete; Rehabilitation; Carbon fiber-reinforced polymers (CFRP); Higher mode effects; Shake table; Seismic performance.

Introduction

RC shear walls are widely used as the seismic force resisting system in medium- and high-rise buildings. In the past few decades, there have been considerable advancements in the analysis and design of RC walls for new construction. The newly adopted performance evaluation methodology and capacity design principles are examples of these important advancements in seismic engineering. Therefore, there is an essential need to upgrade the seismic performance of existing RC shear walls such that they can meet the recent requirements of the design codes.

Tremblay et al. (2001) indicated the importance of higher mode effects for slender high-rise shear walls that result in higher shear

forces and bending moments in the upper region of the wall. The analytical study showed that these higher demands would lead to the formation of a plastic hinge at that upper region. Similar conclusions were reported by Bachmann and Linde (1995), Priestley and Amaris (2002), and Panneton et al. (2006). For experimental validation, two identical scaled (1:0.429) 8-story RC shear walls were constructed and tested on the shake table of École Polytechnique de Montréal (EPM). The tests aimed to investigate the higher mode effects on multistory RC shear walls subjected to eastern North America ground motions that are expected to be rich in high frequencies (Ghorbanirenani et al. 2010). The walls were built with a total height of 9.0 m, and they were designed according to the National Building Code of Canada (NBCC) (2005) and the Canadian Standard CSA-A23.3 (2004) as moderately ductile walls. The walls were designed based on the codes' regulations that a single plastic hinge would occur at the wall base. On the other hand, the nonlinear dynamic analysis of the code-designed walls predicted inelastic response at higher levels of the wall. The shake table tests on the original walls showed the formation of another plastic hinge at the 6th-story level in addition to the one at the wall base. These findings raised the fact that there is a need for retrofitting RC walls that are susceptible to increased demands at their higher level.

Previous research investigated several retrofit techniques using different materials for upgrading the seismic performance of RC shear walls. These ranged from using steel, concrete, fiber-reinforced polymer (FRP) composites, 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; Effendy et al. 2006). The target of intervention for retrofit techniques varied from increasing the strength of the wall, its ductility, or both. In general, it is important to assess the response of an existing RC

¹Ph.D. Candidate, Dept. of Building, Civil and Environmental Engineering, Concordia Univ., Montréal, Québec, Canada, H3G 1M8. E-mail: h_elsokk@encs.concordia.ca

²Associate Professor, Dept. of Building, Civil and Environmental Engineering, Concordia Univ., Montréal, Québec, Canada, H3G 1M8 (corresponding author). E-mail: galal@bcee.concordia.ca

³Engineer, Civil Engineering Dept., École Polytechnique de Montréal, P.O. Box 6079, Montréal, Québec, Canada, H3C 3A7. E-mail: iman-2.ghorbani-renani@polymtl.ca

⁴Professor, Civil Engineering Dept., École Polytechnique de Montréal, P.O. Box 6079, Montréal, Québec, Canada, H3C 3A7. E-mail: pierre.leger@polymtl.ca

⁵Professor, Civil Engineering Dept., École Polytechnique de Montréal, P.O. Box 6079, Montréal, Québec, Canada, H3C 3A7. E-mail: robert.tremblay@polymtl.ca

Note. This manuscript was submitted on December 18, 2011; approved on May 24, 2012; published online on May 28, 2012. Discussion period open until July 1, 2013; separate discussions must be submitted for individual papers. This paper is part of the *Journal of Composites for Construction*, Vol. 17, No. 1, February 1, 2013. © ASCE, ISSN 1090-0268/2013/1-79-90/\$25.00.

wall to identify the most suitable retrofitting technique that meets a target performance.

FRP composite materials are becoming widely proven as an effective alternative for seismic retrofit of RC structures. This is primarily because of their high strength-to-weight ratio, high resistance to corrosion, and the ease of application. FRP sheets were used to increase the flexural capacity, shear capacity, or both flexural and shear capacities of deficient RC walls by having different orientation of the sheets. Lombard et al. (2000) increased the flexural capacity, stiffness, and the shear capacity of RC shear walls by applying one horizontal layer of carbon FRP (CFRP) sheet that is sandwiched between two vertical layers of CFRP. The vertical sheets were anchored to the foundation using steel angles. The researchers reported that FRP-retrofitted walls showed an improved performance provided that a proper anchorage system for the FRP sheets is used. They noted also that premature debonding of FRP sheets due to the compressive stresses in the vertical FRP laminates is a critical issue in case of cyclic loading and that should be avoided. Paterson and Mitchell (2003) retrofitted RC shear walls using CFRP wraps and through-thickness headed bars. The retrofit scheme aimed to increase the shear strength of the wall and to provide additional confinement to the wall's end columns. The FRP-retrofitted wall was able to reach a displacement ductility level that is 57% higher than that of the control wall and was able to dissipate three times the energy absorbed by the original wall. Antoniadis et al. (2003) used vertical FRP strips at the wall edges and horizontal FRP jackets to increase the wall flexural and shear capacities, respectively. They examined different anchoring systems of the vertical FRP sheets including the use of glass FRP (GFRP) or steel anchors. Khalil and Ghobarah (2005) increased the flexural ductility of RC walls by applying biaxial FRP sheets as well as horizontal FRP U-wraps at the end columns, and they were anchored to the wall using either steel or FRP anchors. The rehabilitated walls were able to reach higher displacement ductilities than those of the control wall.

In this study, the original 8-story RC walls tested on the shake table (designated as W1 and W2) experienced severe cracking and yielding of flexural reinforcement at the 6th-story level owing to the higher mode effects. This part of the wall was not initially designed and detailed for such demands [although the design followed the NBCC (2005) and CSA A23.3 (2004) requirements]; and hence, it needs to be rehabilitated. The damaged walls W1 and W2 were rehabilitated and designated as W1R and W2R. Given the response of the wall, the rehabilitation strategy was designed to increase the flexural and shear capacities of the wall section at the 6th-story level of the wall. On the other hand, the wall base was initially designed to act as a plastic hinge region. Because the demands on this location did not exceed the wall capacity, there was no need to increase the wall flexural capacity at the wall base, and only the wall ductility capacity needed to be enhanced.

The objective of this study is to evaluate the effectiveness of using CFRP composites for retrofitting existing RC shear walls that are susceptible to increased demands at upper floors, which were not accounted for in their initial design and detailing, owing to the effect of higher modes of vibration.

Original Walls W1 and W2

The purpose of the shake table tests on the original walls was to investigate the effect of higher modes of vibrations on the seismic performance of slender RC walls. Two identical RC walls, W1 and W2, were constructed using a prototype 8-story residential building with a total height of 20.95 m. The building is located in Montréal,

Québec, and it was scaled by a length factor of 0.429 (Ghorbanirenani et al. 2012). The walls were designed according to NBCC (2005) and CSA-A23.3 (2004) as moderately ductile walls (with a ductility-related force modification factor $R_d = 2.0$ and over-strength-related force modification factor $R_o = 1.4$) and assuming a site Class C. The model wall has a rectangular section with a total height of 9.0 m, length of 1,400 mm, a thickness of 80 mm, and a change in the width at the 6th story as shown in Fig. 1. The walls were constructed using concrete with specified compressive strength, f'_c , of 30 MPa. The flexure steel yield strength, f_y , was measured on average to be 455 MPa, and the shear reinforcement yield strength was 496 MPa. D3.0 deformed bars of cross-sectional area of 19.6 mm² were used for the shear reinforcement and the hoops. The hollow rebars shown in Fig. 1 are longitudinal steel reinforcement bars that were debonded (no adherence) because their contribution to the wall's flexural resistance was not required in the wall design. However, they are required to provide confinement for the boundary zones as required by CSA-A23.3 (2004) for moderately ductile walls. The uniaxial seismic simulator of EPM has a payload capacity of 150 kN and 3.4 m × 3.4 m plan dimension. Owing to the limited vertical capacity of the shake table, it was decided to install the 60 kN seismic weight of each floor beside the shake table in front of each floor level on four multilevel hinged posts as shown in Fig. 1(a and b). Rigid steel beams were connecting the masses to the wall at each floor level to transfer the inertia forces. The axial load carried by the tributary area of the wall was simulated using pretensioned tendons that are anchored to the wall base and top. A value of axial load of 90.7 kN was applied on the wall. The as-built walls are shown in Fig. 1(c), whereas Fig. 1(d) shows the wall cross sections and reinforcement details at the base and the 6th story. Details of the test setup can be found in Ghorbanirenani et al. (2012).

Selected Ground Motion

The ground motion selected for the original and rehabilitated walls was a simulated excitation of an M7.0 event at 70 km from Montreal and modified in the frequency domain to match the NBCC (2005) design spectrum of Montreal. The ground motion acceleration was scaled with an intensity scale factor of 2.65 for the similitude requirements. The scaled ground motion has a peak ground acceleration (PGA) of 0.92_g, a peak ground velocity (PGV) of 0.42 m/s, and an acceleration/velocity ratio (A/V) of 2.19. The details about the loading protocol for the original walls can be found in Ghorbanirenani et al. (2012). Similar to the original wall test protocol, the FRP-retrofitted walls were subjected to sequential intensities of the same ground motion as follows: 100 and 120% of the design ground motion for the first rehabilitated wall (W1R); and 100, 120, 150, and 200% of the design ground motion for the second rehabilitated wall (W2R). Before each application of the ground motion excitation, an impact test was conducted to examine the dynamic properties of the wall. Fig. 2 shows the acceleration time history for the selected ground motion and its response spectrum.

Behavior of the Original Walls W1 and W2

The moderately ductile RC walls (W1 and W2) experienced major cracking and yielding of flexure reinforcement at the 6th-story level because of higher demands than those stated in the design code. The wall section at the 6th-story level was subjected to a factored moment demand, M_f , of 137 kN·m and 132 kN·m for wall W1 and W2, respectively, when subjected to 100% of the design ground motion intensity. These values exceeded the factored flexural resistance of the wall section at the 6th-story level, M_r , which was

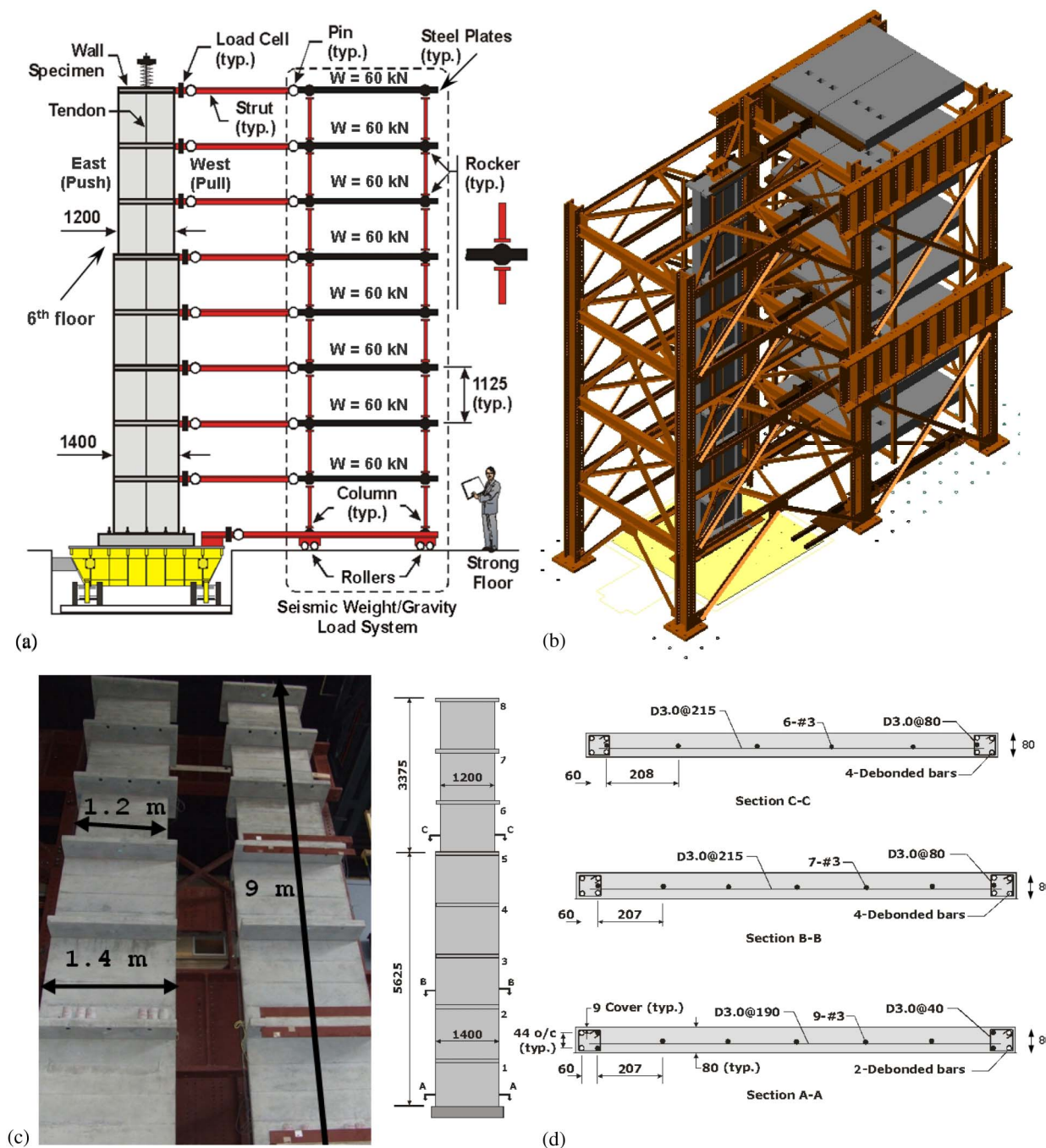


Fig. 1. (a) Test specimen and seismic weight/gravity load system; (b) complete test setup with stabilizing steel frame; (c) model wall; (d) cross section of model wall

calculated to be 118 kN·m. The rotational demand and strains of the flexure reinforcement at the 6th-story level were higher than those at the wall base (plastic hinge region) when the walls were subjected to 100% of the design ground motion intensity (Ghorbanirenani et al. 2012). No concrete spalling occurred at any location. The shear reinforcement did not yield at any location at the 100% intensity. More details about the original wall behavior will be explained in the “Results and Discussion” section.

Rehabilitation Strategy

The target of rehabilitation is to enhance the seismic behavior of the original wall at the two locations that experienced nonlinear

response; at the highest demand location at the base of the wall and at the 6th-story level that experienced a moment demand higher than the design one. As the cause of damage in the two aforementioned locations of the wall is different, the retrofit scheme was not the same. At the 6th-story level, the retrofit strategy was to increase the flexural capacity of the wall section at that level by applying vertical CFRP sheets at the boundary zones of the wall. As a result of increasing the wall’s flexural capacity at that level, the retrofit scheme must consider increasing the shear capacity of the panel to respect the capacity design philosophy such that the FRP-retrofitted wall would not fail in shear before reaching its increased flexure capacity. Therefore, in addition to the vertical CFRP sheets at the wall edges, horizontal CFRP wraps were applied to increase the shear capacity of the wall section at the 6th story. Therefore,

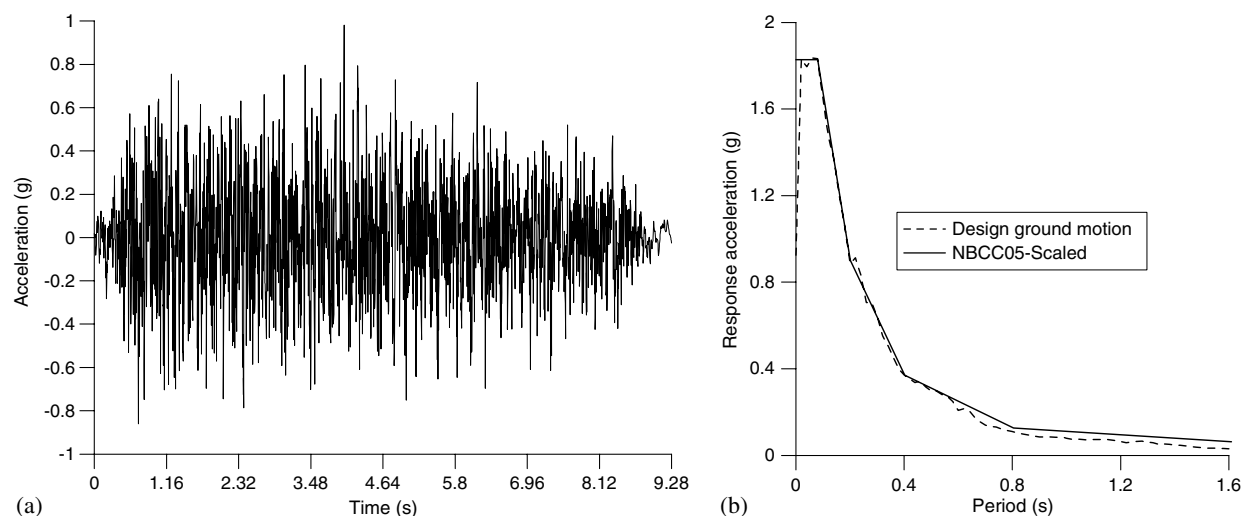


Fig. 2. Selected ground acceleration: (a) time history; (b) response spectra

this rehabilitation scheme will increase the wall flexural and shear strengths, and it will provide confinement at the boundary zones of the 6th-story level. As for the plastic hinge location at the base of the wall, only horizontal CFRP wraps were applied to improve the confinement at the boundary zones, thus increasing the wall's ductility without increasing its flexural strength.

Rehabilitated Wall W1R

Because the original walls did not experience concrete spalling at any location, no concrete replacement was needed. The wall surface was cleaned and ground in several areas to achieve a smooth surface. The wall corners were chamfered to a radius of 10 mm to avoid stress concentration when wrapping the FRP sheets. Because of excessive yielding of the flexural reinforcement measured at the 6th story of the two original walls, resulting from higher demands than those stated in the design codes, the rehabilitation scheme necessitates increasing the wall's flexural capacity at that level. This was achieved by applying a 200-mm wide vertical unidirectional CFRP strip at the wall extremities on both faces. Table 1 shows the mechanical properties of the Tyfo SCH-11UP composite system; dry fiber, TyfoS epoxy, and CFRP composite (Fyfe 2010) used in the rehabilitation process. The width of the vertical CFRP strips were designed such that the factored flexural resistance of the rehabilitated wall section at the 6th-story level becomes greater than the factored moment demand observed in the tests on the original walls (137 kN·m). A design value of 157 kN·m was then chosen to account for the increase in wall demands because of its increased stiffness arising from the added flexural CFRP reinforcement. The ultimate strain of the FRP vertical sheets 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 CFRP sheets.

The vertical FRP strips were anchored to the top and bottom slabs of the 6th-story panel using FRP fan anchors as shown in Fig. 3. The FRP anchors were placed in previously drilled holes that were filled with epoxy resin. The drilled holes made an angle of 30° with the wall surface, and they went through the wall web and slab for a distance at least 60 mm as shown in Fig. 4. The holes were cleaned and washed to remove any dust before the installation of FRP anchors to ensure a high bond between the resin and concrete substrate.

To ensure a ductile performance after increasing the wall's flexural capacity, the shear capacity at the 6th story was increased by applying one horizontal layer (for each face of the wall) of C-shaped CFRP sheet on top of the vertical FRP strips. The two C-shaped FRP sheets were overlapped at the boundary regions of the wall to provide a better confinement of the wall end columns as shown in Fig. 3. The horizontal CFRP wraps also help in reducing the tendency of premature debonding of the vertical strips under compression during the cyclic loading as was pointed out by Lombard et al. (2000) as an unfavorable response that needs to be avoided.

The horizontal CFRP sheets were anchored along the sides of the wall using previously drilled through-thickness steel anchors of 10 mm diameter and spaced vertically at 210 mm. The steel anchors are expected to result in a higher confinement of the wall end columns by providing a lateral support for the FRP wraps at certain intervals, which would enhance the added confinement provided by the CFRP wraps. The steel anchors were used in lieu of FRP fan anchors (spike type) based on the findings of Khalil and Ghobarah (2005). They reported that steel anchors did not fail in shear as was the case for FRP anchors, and that the rehabilitated wall using steel anchors was able to sustain a higher load capacity.

At the base of the wall, no additional flexural strength was needed. Therefore, no vertical FRP strips were used at the base story. The base panel was wrapped horizontally using the C-shaped CFRP sheets (one at each face) and anchored to the wall using the through-thickness steel anchors, similar to the 6th story. The horizontal wrapping would confine the wall boundary regions, thus increasing the wall ductility and energy dissipation capacity.

For the first rehabilitated wall, W1R, the ground motions were applied at two intensity levels; 100 and 120% of the design ground

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

Parameter	Typical dry fiber	Epoxy material	CFRP composite
Tensile strength (MPa)	3,790	72.4	903
Elongation at break (%)	1.60	5.00	1.05
Tensile modulus (GPa)	230	3.18	86.9
Laminate thickness (mm)	0.175	Not applicable	0.27

Note: Data from Fyfe (2010).

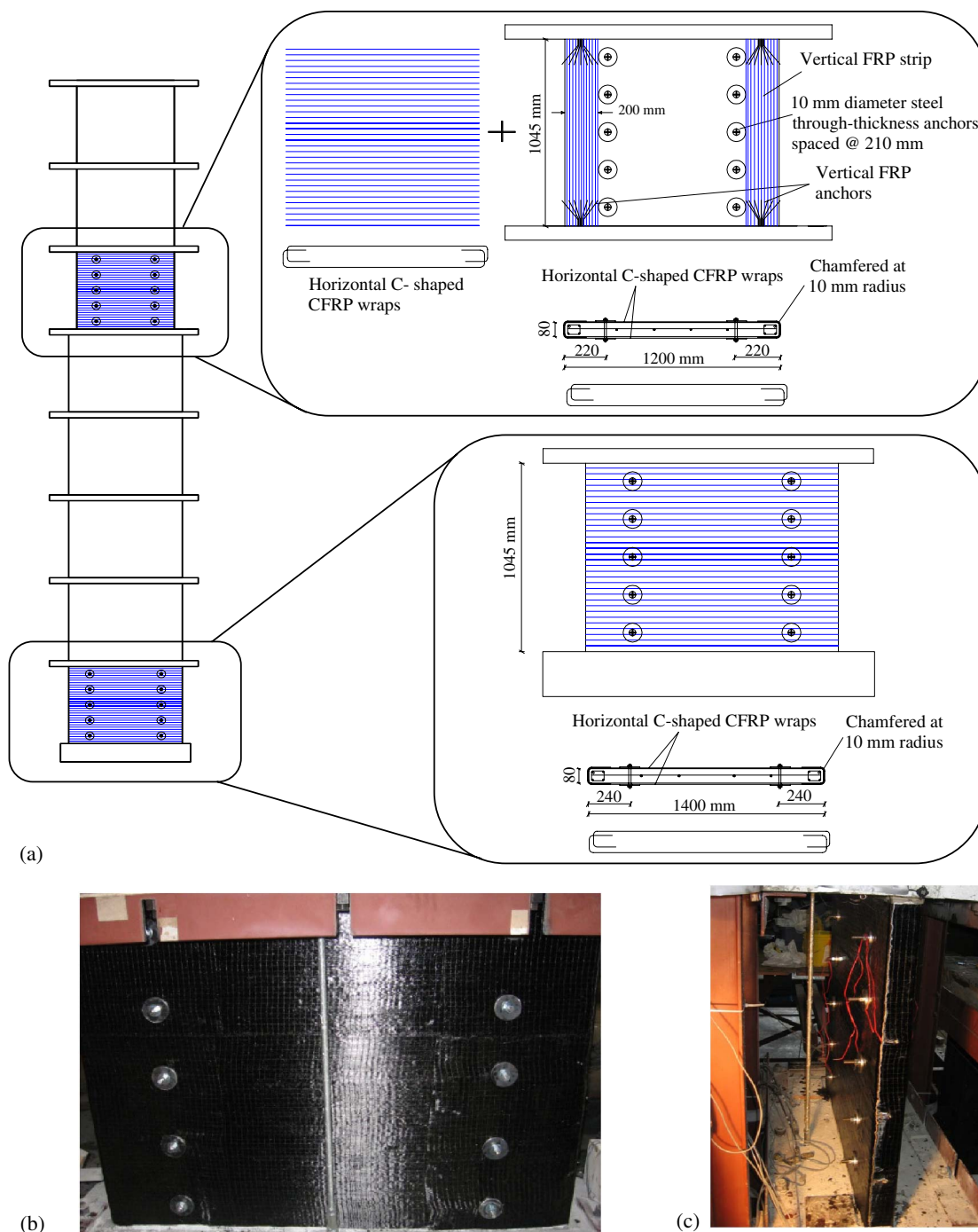


Fig. 3. (a) Rehabilitation scheme of W1R; (b) base panel rehabilitation; (c) 6th-story panel rehabilitation

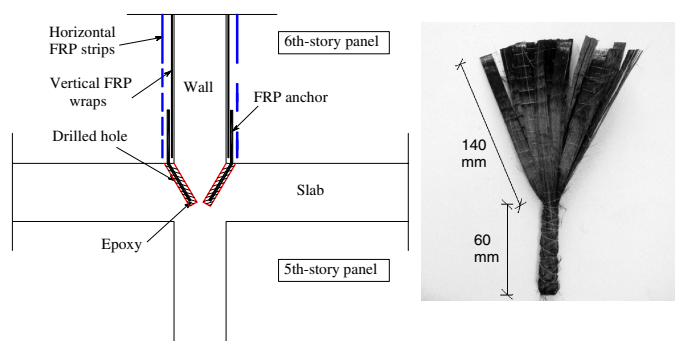


Fig. 4. Detail of CFRP anchor for the first rehabilitated wall, W1R

motion intensity. Similar to the tests on the original walls, impact tests were carried out before each application of the ground motion level and at the end of the tests to determine the dynamic characteristics of the tested walls and to estimate the amount of damage occurred.

Rehabilitated Wall W2R

For the second rehabilitated wall W2R, a rehabilitation scheme similar to W1R was used for both the base and 6th-story panels. Fig. 5 shows the rehabilitation scheme of wall W2R. For this wall, it was decided not to use the through-thickness steel anchors. The FRP anchors for W2R were detailed in a different way than W1R

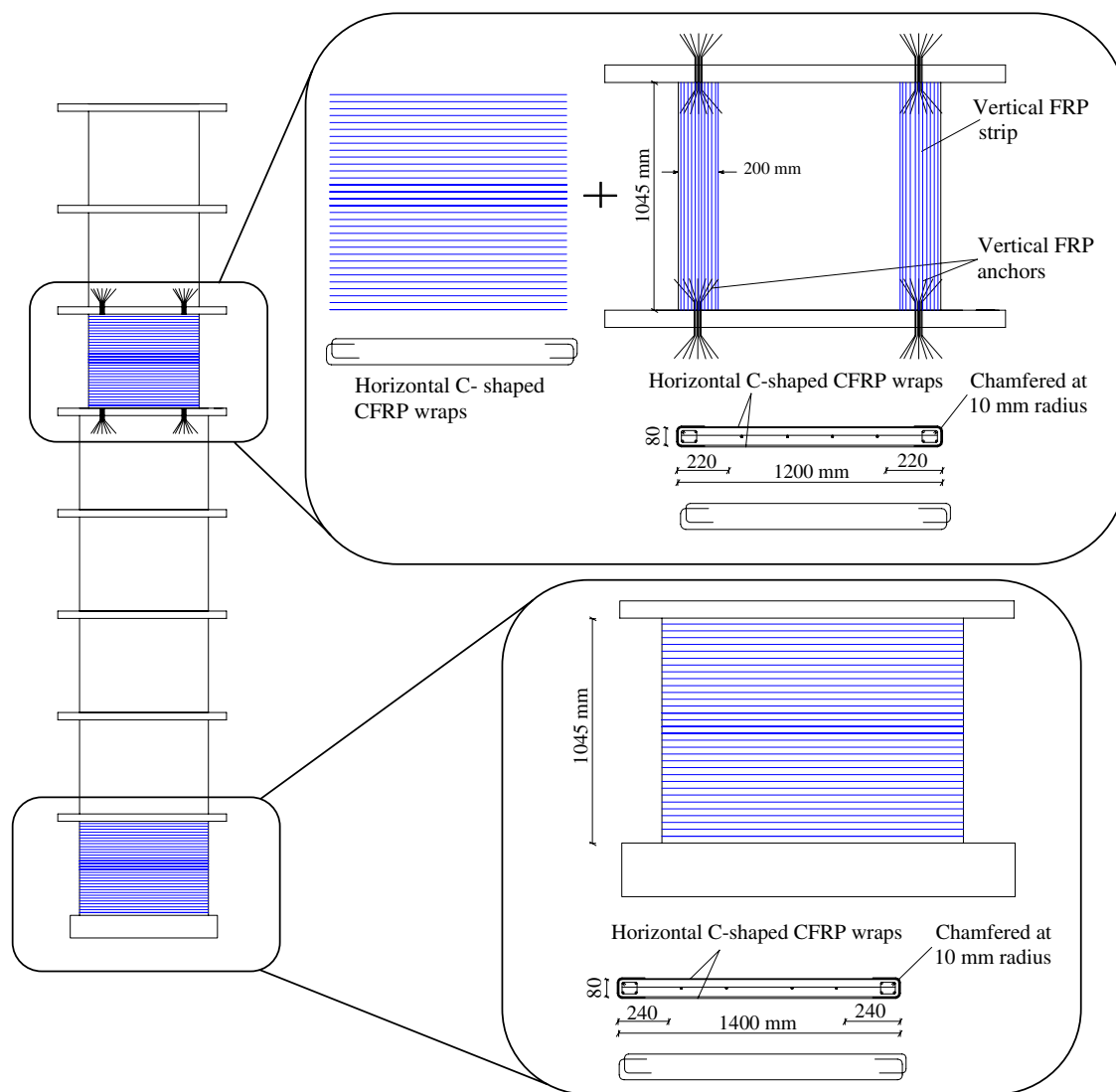


Fig. 5. Rehabilitation scheme of W2R

owing to the difficulty experienced in drilling the inclined holes through the wall web for W1R. This was because the wall reinforcement was concentrated at the wall boundary columns where the FRP anchors should be. Therefore, it was decided to investigate a different technique for detailing the FRP anchor of the second rehabilitated wall W2R. As the amount of reinforcement in the slab is less than the wall web at the wall extremities, it was decided to drill the anchor holes through the slabs. The holes were drilled through the slab thickness with a slight inclination as shown in Fig. 6. This detailing also allowed a longer embedment length of the FRP anchor to reduce the probability of anchor debonding from the concrete substrate. For the rehabilitated wall, W2R, the ground motion was applied at four intensity levels: 100, 120, 150, and 200% of the design intensity, similar to the original wall test.

Design and Fabrication of CFRP Anchors

The FRP fan anchors were prepared using the same dry fibers and epoxy resin of the CFRP composite system used in the rehabilitation. The dry fibers weigh 3.1 N/m^2 with a nominal thickness of 0.175 mm/ply . Each FRP anchor used for wall W1R was made from an 80-mm wide sheet that was cut into 8 strips each of 10-mm width and 400-mm length. The strips were folded into

two at its midlength and tied at the folded end to form an anchor with an embedment length of 60 mm and a fan length of 140 mm as shown in Fig. 4. For wall W2R, each FRP anchor was made using a 160-mm wide sheet that was cut into 16 strips each of 10 mm width and length of 360 mm. The strips were tied together at the middle as shown in Fig. 6 to form an anchor with a double fan of 140-mm length and embedment length of 80 mm (represents the slab thickness). The anchor diameters ranged from 10–12 mm. The total amount of fibers in the cross section of the FRP anchors used for walls W1R and W2R was the same, and they were expected to reach the same ultimate capacity. The FRP anchors were designed to have an ultimate capacity equal to 35% of the ultimate strength of the straight fiber composite to take into account the strength reduction resulting from the bent fibers. The design capacity of the FRP anchor (using ultimate strain of 0.0037 mm/mm) is 16.5 kN. This value was checked using the coupon tests carried by Ozbakkaloglu and Saatcioglu (2009) in which they tested 81 CFRP anchors under direct pullout. The chosen 16.5 kN anchor design capacity was less than the values reported by Ozbakkaloglu and Saatcioglu (2009) for the same fiber strip width and the 53-mm embedment length. Therefore, the capacity of the two FRP anchors used for each vertical CFRP strip was equal to 33.0 kN. This value exceeds the nominal tensile strength of the vertical CFRP strip

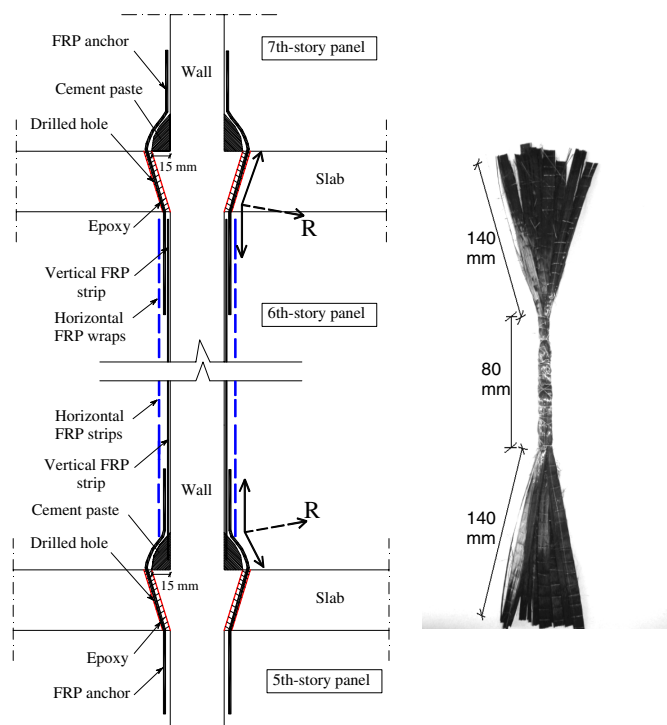


Fig. 6. Detail of CFRP anchor for the second rehabilitated wall W2R

when calculated using its design strain of 0.006 and the typical test value provided by the manufacturer. This would ensure the rupture of vertical CFRP strips before the failure of CFRP anchors.

Instrumentation

Similar to the original wall tests, floor accelerations were measured using accelerometers attached to each floor and to the shake table. The horizontal movement of each floor in relationship to the shake table movement was measured by using horizontal cable-extension

position transducers (CEPTs) that are attached to each floor and to a reference wall. The longitudinal and diagonal deformations of the wall base and the 6th-story panels were measured using vertical and diagonal CEPTs. In addition, three vertical linear variable differential transducers (LVDTs) were attached to the two wall extremities at both levels to measure the wall curvature. The strain gauges attached to the original walls' reinforcement were used, and the damaged ones were disconnected from the data acquisition system. Additional strain gauges were applied to the CFRP sheets at different locations with different orientation. A total of 32 strain gauges with a gauge length of 5 mm were recording the strains of steel reinforcement and FRP sheets at different locations along the wall height. Their locations at the base and 6th-story panels are shown in Fig. 7.

Results and Discussion

Dynamic Characteristics

Impact tests on the wall were conducted before and after retrofitting the walls and before each application of the ground motion to assess the variation in the dynamic characteristics of the wall. The change in the wall periods of vibration is used as an indicator of the global damage in the wall (in case of period elongation), or on the other hand, as an indication of seismic upgrade and stiffening (in case of period shortening).

Table 2 shows the periods of vibration for the first and second tested walls. The table shows the first two periods of vibration before and after testing the original walls (W1 and W2) and before and after testing the rehabilitated walls (W1R and W2R). The table shows the elongation of first and second periods of vibration for the original walls after the test as a result of their stiffness degradation resulting from the accumulated incurred damage. After FRP-retrofit, the first and second periods of vibration of the two retrofitted walls decreased (compared with that of the damaged original walls), which indicates that the FRP rehabilitation scheme increased the wall's stiffness. This is because of the application of the vertical FRP strips at the 6th-story level which increased the

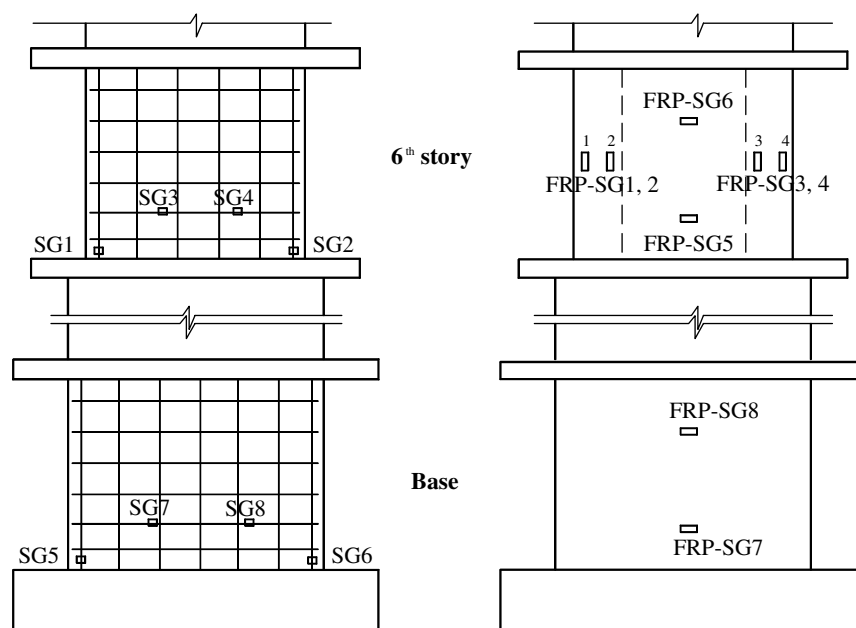


Fig. 7. Locations of the strain gauges installed on steel reinforcement and FRP sheets at the base and 6th-story panels

Table 2. Periods of Vibration of the Original and Retrofitted Walls before and after Application of 100% of the Ground Motion Intensity

	First wall		Second wall	
	T_1 (s)	T_2 (s)	T_1 (s)	T_2 (s)
Time	Original wall W1		Original wall W2	
Before	0.67	0.14	0.65	0.14
After	0.96	0.24	1.31	0.36
	Retrofitted wall W1R		Retrofitted wall W2R	
Before	0.76	0.19	0.69	0.17
After	1.09	0.23	1.00	0.28

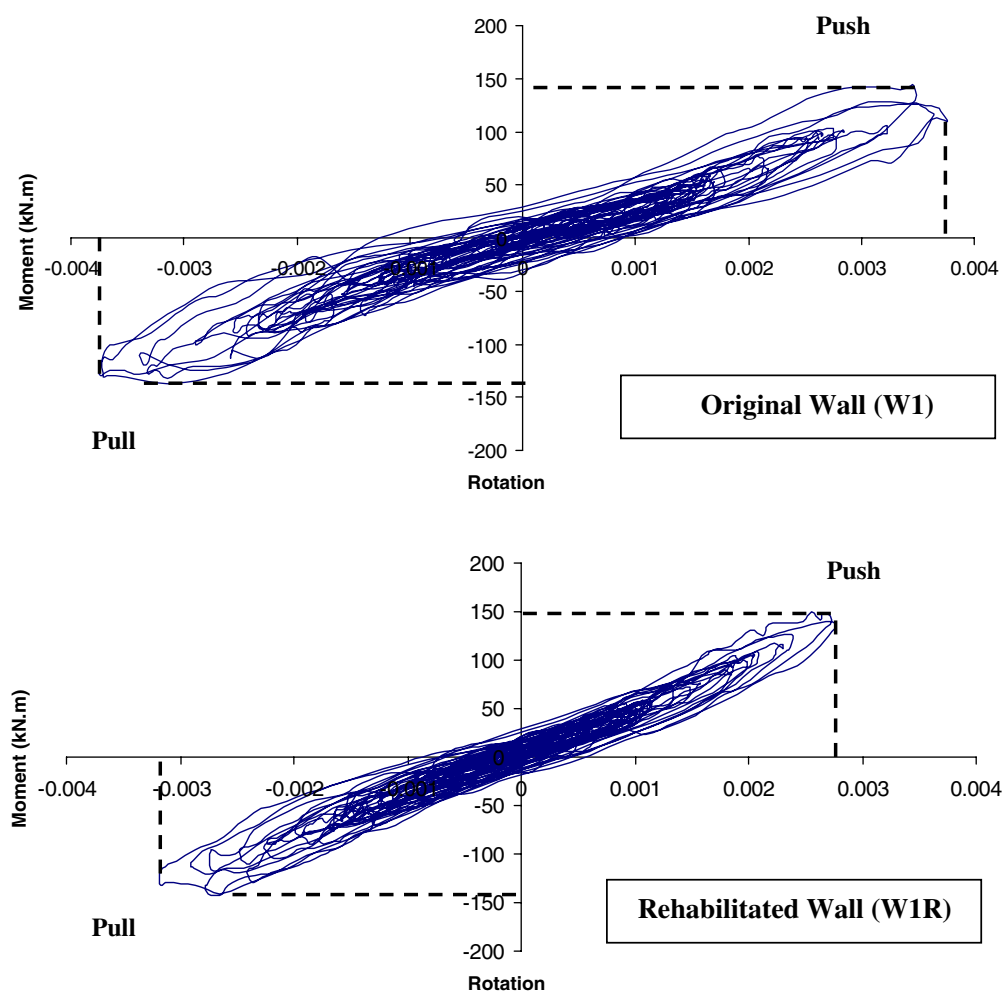
wall's stiffness. After the retrofitted walls were subjected to the selected ground motion records, the walls' first and second periods of vibration increased again owing to the damage that occurred.

Story Forces and Deformations

Fig. 8 shows the moment-rotation relationship at the 6th-story level for the first original and rehabilitated walls (i.e., W1 and W1R) when subjected to 120% of the ground motion intensity. Fig. 9 shows the same relationship for the second wall (i.e., W2 and W2R) when subjected to 200% of the ground motion intensity.

Fig. 9 shows that at a higher level of the ground motion intensity, the original wall was already damaged, and unsymmetric rotational demands were observed. This can be attributed to the large pulse from the shake table that occurred at the beginning of the ground motion record as shown in Fig. 2 (at time = 0.7 s), which pulled the wall toward the west side. Figs. 8 and 9 show that the added vertical FRP strips led to a reduction in the maximum story rotation by approximately 28% for the first wall at 120% intensity and by 26% for the second wall at 200% intensity, whereas the walls' moment demands at the 6th-story level have slightly increased. This implies that the rehabilitated walls at the 6th-story level were able to resist higher force demands with less rotational ductility demands. This shows the effectiveness of the FRP retrofit scheme used at the 6th-story panel.

Tables 3 and 4 summarize the rotational demands (θ), the shear force (V), and overturning moment (M) demands at the base level and the 6th-story level for the original walls (W1 and W2) and the rehabilitated walls (W1R and W2R) at different ground motion intensities, respectively. Similar conclusions can be derived from Tables 3 and 4 when comparing the rotational demands at the 6th-story level before and after rehabilitation. From the tables, the base shear and overturning moment slightly increased because of rehabilitation. This is attributed to the additional stiffness at the 6th-story level resulting from the application of the vertical FRP strips, which led to the reduction of the wall periods of vibration as shown in Table 2.

**Fig. 8.** Moment-rotation relationship at the 6th-story level for the first control and rehabilitated wall (W1 and W1R) when subjected to 120% of the ground motion intensity

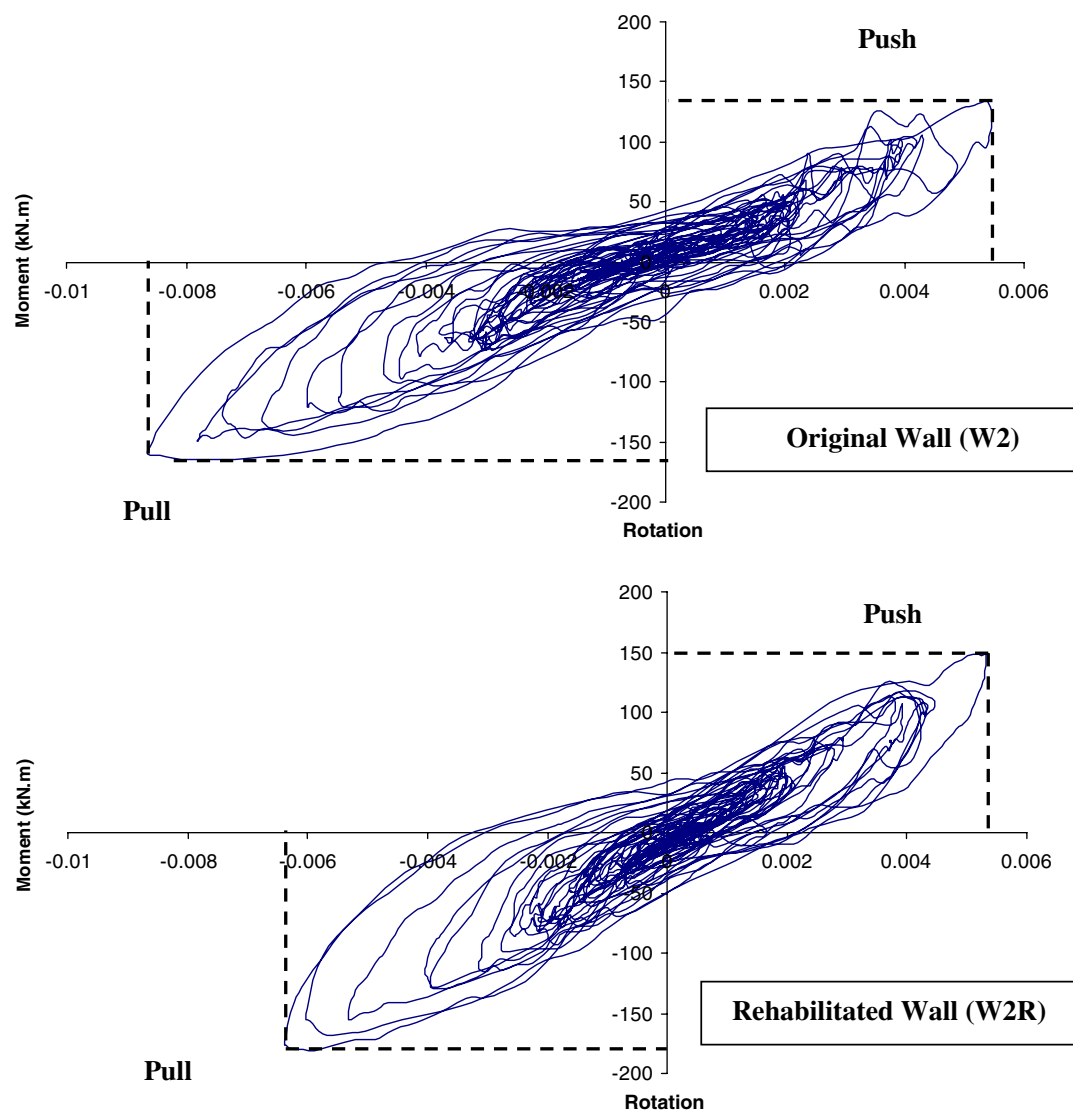


Fig. 9. Moment-rotation relationship at the 6th-story level for the second control and rehabilitated wall (W2 and W2R) when subjected to 200% of the ground motion intensity

Table 3. Test Results of the First Original and Rehabilitated Walls, W1 and W1R

Measured Parameter	Level	100% intensity		120% intensity	
		W1	W1R	W1	W1R
θ (10^{-3} rad) ^a	6th	2.84	2.3	3.77	3.17
	Base	2.43	1.89	2.58	2.62
V (kN) ^b	6th	40.2	39.3	46.2	43
	Base	118	131	139	156
M (kN·m) ^c	6th	137	118	144	150
	Base	241	259	261	274

^aRotational demand.

^bStory shear force.

^cStory overturning moment.

Table 4. Test Results of the Second Original and Rehabilitated Walls, W2 and W2R

Measured Parameter	Level	120% intensity		150% intensity		200% intensity	
		W2	W2R	W2	W2R	W2	W2R
θ (10^{-3} rad) ^a	6th	2.78	3.17	5.82	4.51	8.61	6.39
	Base	2.54	2.60	3.59	3.35	3.72	3.83
V (kN) ^b	6th	45	41.5	46	40	65	53
	Base	140	161.9	172	172	183	202
M (kN·m) ^c	6th	141	145	157	167	165	181
	Base	225	237	243	253	253	255

^aRotational demand.

^bStory shear force.

^cStory overturning moment.

Strains of Steel Reinforcement and FRP Sheets

Tables 5 and 6 show the strains of the longitudinal and transverse steel reinforcement and the vertical and horizontal FRP sheets for the first and second walls, respectively. The shown values are for the original walls (W1 and W2) and the rehabilitated walls (W1R and W2R) when subjected to two different ground motion intensities.

For the first original wall, W1, most of the strain gauges attached to the steel reinforcement were still functioning after testing, which allowed their use in the tests on the rehabilitated wall, W1R. No yielding of the shear reinforcement occurred at the wall base or the 6th-story level.

For the longitudinal steel reinforcement strains, Table 5 indicates that a significant reduction of longitudinal reinforcement strains at the 6th-story level occurred primarily because of the application of the vertical FRP strips at that level which led to the reduction of the wall rotational demands as was mentioned in the previous section. On the other hand, the strain values of the longitudinal steel reinforcement at the wall base did not decrease as no vertical FRP sheets were applied at the base during rehabilitation. The vertical FRP strips reached a strain of 2,000 μ strain (almost 33% of the FRP material's design capacity) at the 120% of the ground motion intensity. No FRP anchorage failure or debonding was detected.

From Tables 3 and 5 and considering the 120% of the ground motion intensity, the strain of the longitudinal reinforcement at the 6th-story panel of the original wall W1 was shown to reach a value of 9,800 μ strain when the wall factored moment demand, M_f , was 144 kN·m. This factored moment demand exceeded the wall factored flexural resistance, M_r , at that level which was calculated to be 118 kN·m. Using section analysis of the original wall, the strains of the longitudinal reinforcement would be 3,300 μ strain at $M_f = M_r = 118$ kN·m. The main design target of the FRP-retrofit scheme is to increase the factored moment resistance of the 6th-story panel without increasing the strains of its flexure steel reinforcement. For W1R, the FRP rehabilitation was able to reduce the strains in the longitudinal steel reinforcement (at higher moment demand $M_f = 150$ kN·m) to 3,250 μ strain when subjected to 120% of the design ground motion intensity, which is less than the strain value corresponding to the factored moment resistance of wall W1 (3,300 μ strain). This shows the efficiency of the FRP-rehabilitation scheme in achieving the primary design target even at 120% of the design ground motion intensity.

Table 5 also shows that the horizontal CFRP sheets applied at the wall base (where the high shear force demands were observed) were able to reduce the strains in the transverse steel reinforcement at that location. On the other hand, the strains of the transverse reinforcement at the 6th-story level were insignificant. The strains of the horizontal CFRP sheets did not reach high values as the wall behavior was primarily governed by flexure. Nevertheless, the horizontal FRP sheets are still needed in case the retrofitted wall's flexural capacity would exceed its original shear capacity, in addition to their primary function in confining the wall's boundary elements, and hence increase the wall ductility at the base.

For the second original wall W2, the capacity of most of the strain gauges installed on the flexure reinforcement at the wall base and at the 6th-story level were exceeded owing to excessive yielding that occurred when the original wall was subjected to 200% of the design ground motion intensity (Table 6). The shear reinforcement at the wall base has yielded ($\varepsilon_s = 2,500$ μ strain) when the wall was subjected to 200% of the design ground motion intensity, whereas at the 6th-story level, the shear reinforcement strains were insignificant. From the table, the strains in the horizontal FRP sheets were not significant at the base panel; however, they were able to reduce the strains of the transverse steel reinforcement at that level by almost 55%.

Importance of Anchorage Detailing

Figs. 4 and 6 show the two different FRP anchorage detailing used for the rehabilitated walls W1R and W2R, respectively. For the second rehabilitated wall (W2R), it was expected that the CFRP vertical strips would reach higher strains when the wall was subjected to 200% of the ground motion intensity than the strains reached for W1R at 120% intensity. Tables 5 and 6 show that the vertical CFRP strips for W2R (at 200% of ground motion intensity) reached strain values that were even less than those of W1R (at 120%). In addition, by comparing the moment-rotation curves for W2 and W2R in Fig. 9, the CFRP sheets reduced the wall rotation only for the pull cycles (where the wall rotation reduced from 0.0086 to 0.0065). During the push cycles, the wall rotation almost did not change before and after rehabilitation. This indicates that the vertical FRP strips were not anchored properly to the slabs on the wall's west side. On the other hand, Fig. 8 shows that the vertical CFRP strips applied to the 6th-story panel of the first wall, W1R, were able to reduce the wall rotation in both push and pull directions. This indicates the importance of anchorage detailing in the efficiency of FRP flexure rehabilitation.

During the tests on the second rehabilitated wall (W2R), the horizontal CFRP wraps at the 6th-story panel were not able to prevent the FRP anchor at the bottom slab from stretching outward from the wall while being tensioned (Fig. 6). Consequently, part of the CFRP anchor strain was lost during this undesired elongation, and this resulted in small strains in the vertical CFRP strips. The CFRP anchor connected to the top slab was efficient as the force resultant in the FRP anchor (R) was acting against the

Table 5. Strains of Steel Reinforcement and the FRP Sheets for the First Wall (Microstrain)

Panel location	Strain gauge location	100% intensity		120% intensity	
		W1	W1R	W1	W1R
Base panel	Longitudinal steel reinforcement	2,350	2,380	2,360	2,440
	Transverse steel reinforcement	870	430	1,016	490
	Horizontal FRP sheets	Not applicable	50	Not applicable	50
6th-story panel	Longitudinal steel reinforcement	10,920	3,630	9,800	3,250
	Vertical FRP sheets	Not applicable	1580	Not applicable	2,000

Table 6. Strains of Steel Reinforcement and the FRP Sheets for the Second Wall (Microstrain)

Panel location	Strain gauge location	120% intensity		200% intensity	
		W2	W2R	W2	W2R
Base panel	Longitudinal steel reinforcement	2,500	2,700	11,370	—
	Transverse steel reinforcement	1,440	1,040	2,500	1,130
	Horizontal FRP sheets	Not applicable	106	Not applicable	165
6th-story panel	Longitudinal steel reinforcement	15,100	—	—	—
	Vertical FRP sheets	Not applicable	1,320	Not applicable	1,900

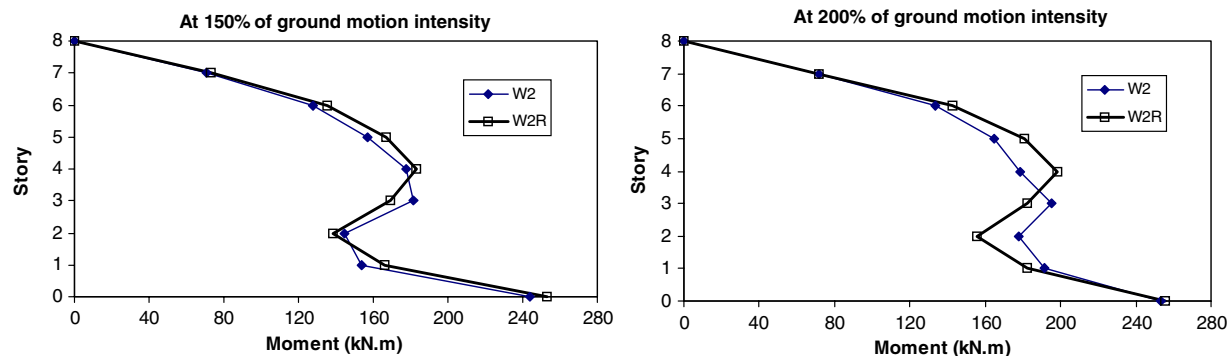


Fig. 10. Effect of FRP-rehabilitation on the wall bending moment demands

concrete slab and not against the horizontal FRP wraps as shown in Fig. 6. Interestingly, the CFRP anchor on the east side of wall W2R was partially efficient during the 200% of the ground motion intensity. This is because the wall experienced a higher rotational demand during the pull cycles, which tends to elongate the CFRP strips on the east side of the wall more than the strips on the west side as shown in Fig. 8 for the second wall. This elongation exceeded the lost elongation in the CFRP anchors that was previously mentioned (outwards stretch), and this resulted in the ability of the CFRP strips to reduce the wall rotation for W2R during the pull cycles as shown in the figure. Therefore, it is recommended to install the FRP anchors as was done for wall W1R despite the difficulty of drilling the holes through the wall web thickness. If the other detail has to be used, the designer has to ensure that the FRP anchor is detailed such that it only elongates in the direction parallel to its axis without any undesired outward deformations that would waste part of the anchor longitudinal strains (i.e., stretch without developing resisting forces).

Effect of FRP-Rehabilitation on the Behavior of Unrehabilitated Panels

Tables 3 and 4 showed that the walls shear force and bending moment demands increased after rehabilitation owing to the vertical CFRP strips applied to the 6th-story panel. Fig. 10 shows the maximum bending moment demands for the second original wall, W2, and the rehabilitated wall, W2R, when subjected to 150 and 200% of the design ground motion intensity. From Fig. 10, the bending moment demands for different stories increased by 3–8% after rehabilitation at the 150% intensity, and by 6–13% for the 200% intensity. This increase in demands would lead to an increase in the wall reinforcement strains after rehabilitation. In this case, because of flexure rehabilitation of one panel of the wall, other panels may need to be rehabilitated if the rehabilitation leads to a significant increase in their strains and deformations. This can be predicted by using a numerical micro- or macromodel, in which the additional stiffness resulting from FRP retrofit can be simulated. This is similar to the work done by Elnady (2008) and Cortes-Puentes and Palermo (2012), in which they simulated the behavior of FRP-retrofitted RC shear walls using macromodeling and finite-element modeling, respectively.

Conclusions

The behavior of two 8-story RC shear walls rehabilitated using CFRP composite sheets was investigated. The walls were tested under the simulated ground motion record using the shake table

at École Polytechnique de Montréal. The original walls were tested to investigate the effect of higher modes on the seismic behavior of multistory RC walls. The rehabilitation strategy aimed at controlling the excessive yielding and deformations that occurred at the higher level (the 6th story), that was not detailed to have plasticity, and enhancing the wall ductility at the wall base. The FRP-rehabilitated walls performed efficiently showing an improved flexural behavior at the 6th-story panel. The FRP rehabilitation reduced the walls' rotations at the 6th story by approximately 28% for the first wall (tested at 120% intensity) and 26% for the second wall (at 200% intensity). The strains of the longitudinal reinforcement at the 6th story decreased significantly after rehabilitation. The FRP-rehabilitated wall was able to achieve the primary design target even at 120% of the design ground motion intensity. The horizontal CFRP sheets were able to enhance the shear behavior of the rehabilitated panels in addition to their function for wall confinement. The walls' shear force and bending moment demands were found to be slightly higher at other floors after rehabilitation, which may require the rehabilitation of these floors. The FRP-rehabilitation scheme would improve the wall seismic behavior provided that a special attention should be taken to the detailing of FRP anchors used for flexure rehabilitation.

Acknowledgments

The authors would like to acknowledge the financial support of le Fonds de Recherche du Québec - Nature et Technologies (FRQNT), le Centre d'Études Interuniversitaire sur les Structures sous Charges Extrêmes (CEISCE), and the Natural Sciences and Engineering Research Council of Canada (NSERC). The authors also gratefully acknowledge Fyfe Co. for donating the FRP composite materials used for 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 Struct. J.*, 100(4), 510–518.
- Bachmann, H., and Linde, P. (1995). "Dynamic ductility demand and capacity design of earthquake-resistant reinforced concrete walls." *Publication SP 157-06, Proc., Tom Paulay Symp.*, American Concrete Institute, Detroit, 117–142.
- Bakis, C., et al. (2002). "Fiber-reinforced polymer composites for construction—State-of-the-art review." *J. Compos. Constr.*, 6(2), 73–87.
- Canadian Standards Association (CSA). (2004). "Design of concrete structures." *A23.3-04*, Mississauga, Ontario, Canada.

- Cortés-Puentes, W., and Palermo, D. (2012). "Modeling of RC shear walls retrofitted with steel plates or FRP sheets." *J. Struct. Eng.*, 138(5), 602–612.
- Effendy, E., Liao, W., Song, G., Mo, Y., and Loh, C. (2006). "Seismic behavior of low-rise shear walls with SMA bars." *Proc., 10th Biennial Int. Conf. on Engineering, Construction, and Operations in Challenging Environments, Earth and Space*, ASCE, Reston, VA.
- Elnady, M. (2008). "Seismic rehabilitation of RC structural walls." Ph.D. thesis, Dept. of Civil Engineering, McMaster Univ., Hamilton, Canada.
- Fiorato, A., Oesterle, R., and Corley, W. (1983). "Behavior of earthquake resistant structural walls before and after repair." *J. Am. Concr. Inst.*, 80(5), 403–413.
- Fyfe. (2010). "Tyfo SCH-11UP composite using Tyfo S epoxy user's guide." Fyfe Co., San Diego, CA, (<http://www.fyfeco.com>) (Oct. 2010).
- 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." *Proc., 9th U.S. National and 10th Canadian Conf. on Earthquake Engineering*, Toronto, ON.
- Ghorbanirenani, I., Tremblay, R., Léger, P., and Leclerc, M. (2012). "Shake table testing of slender RC shear walls subjected to eastern North America seismic ground motions." *J. Struct. Eng.*, 1, 492.
- ISIS. (2008). *ISIS design manual 4—FRP rehabilitation of reinforced concrete structures*, ISIS Canada Corporation, The Canadian Network of Centers of Excellence on Intelligent Sensing for Innovative Structures, Winnipeg, Manitoba.
- Khalil, A., and Ghobarah, A. (2005). "Behavior of rehabilitated structural walls." *J. Earthquake Eng.*, 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." *Proc., 12th World Conf. on Earthquake Engineering* (CD-ROM), Paper No. 2032, Auckland, NZ, New Zealand society for Earthquake Engineering, Upper Hutt, New Zealand.
- National Research Council of Canada. (2005). *National building code of Canada (NBCC)*, 12th Ed., Institute for Research In Construction, Ottawa, ON.
- Ozbakkaloglu, T., and Saatcioglu, M. (2009). "Tensile behavior of FRP anchors in concrete." *J. Compos. Constr.*, 13(2), 82–92.
- Panneton, M., Léger, P., and Tremblay, R. (2006). "Inelastic analysis of a reinforced concrete shear wall building according to the National Building Code of Canada 2005." *Can. J. Civ. Eng.*, 33(7), 854–871.
- Paterson, J., and Mitchell, D. (2003). "seismic retrofit of shear walls with headed bars and carbon fiber wrap." *J. Struct. Eng.*, 129(5), 606–614.
- Priestley, M. J. N., and Amaris, A. (2002). "Dynamic amplification of seismic moments and shear forces in cantilever walls." *Research Rep. ROSE—2002-01*, Rose School of Engineering, Pavia, Italy.
- Taghdi, M., Bruneau, M., and Saatcioglu, M. (2000). "Seismic retrofitting of low-rise masonry and concrete walls using steel strips." *J. Struct. Eng.*, 126(9), 1017–1025.
- Tremblay, R., Léger, P., and Tu, J. (2001). "Inelastic seismic response of concrete shear walls considering P-delta effects." *Can. J. Civ. Eng.*, 28(4), 640–655.