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SHAKE TABLE TESTS AND REPAIR OF DUCTILE SLENDER REINFORCED CONCRETE SHEAR WALLS

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ABSTRACT

The paper describes a shake table test program on two 9m tall reinforced concrete shear wall models that are part of an 8-storey, 20.95m tall building designed in Montreal, QC according to the seismic provisions of the 2005 National Building Code of Canada and the 2004 CSA-A23.3 concrete design standard. The wall is of the moderately ductile category, as commonly built in Eastern North America (ENA). The objective of the test is to examine the contribution of the higher modes to the wall response when subjected to strong ground motions anticipated in (ENA). The focus is on the amplitude and distribution of the horizontal shear force over the building height, the inelastic rotation demand in the upper portion of the wall. It was found that significant inelastic deformations took place in the 6th storey in addition to the base plastic hinge. This dual plastic hinge response is not recognized by current codes. After testing, both walls were rehabilitated using carbon fibre-reinforced polymer (CFRP) composite sheets at the two plastic hinge locations. The walls were then retested using the same shake table loading protocol. At the wall base, uni-directional C-shaped CFRP sheets were applied horizontally on the two long sides of the wall, overlapped at the wall's boundary regions and anchored along the wall sides. On the 6th storey panel, uni-directional CFRP sheets were applied vertically and were anchored to the top and bottom slabs using CFRP anchors, above which uni-directional horizontal C-shaped CFRP sheets were applied. The rehabilitation schemes for the two walls aim to increase the flexural, shear, and ductility capacities of the wall at the 6th storey panel due to the observed increase in demand at that level, whereas the added CFRP confinement at the base panel aimed at increasing the ductility capacity at the wall base. Both rehabilitated walls performed efficiently showing improved flexural strength at the 6th storey panel. Upon increasing the seismic ground motion intensity, the damage (cracking, plasticity) was found to spread in the other unrehabilitated stories.

Introduction

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High rise reinforced concrete (RC) structural walls subjected to severe earthquakes, especially with high frequency content typical of Eastern North America (ENA), behave differently from low rise walls. The higher mode effects significantly change the seismic behavior of these structures. Analytically, contributions of higher modes actions in slender ductile shear walls amplify the base shear and moment demand in the upper part of the walls which cause the formation of plastic hinges at those locations where they were not considered in seismic provision of CSA-A23.3-04 (Tremblay et al. 2008; Ghorbanirenani et al. 2008; Boivin et al. 2008; Panneton et al. 2006, Priestly and Amaris 2002). In complement to analytical results, experimental large scale real time tests are required to investigate the aspects of higher mode contributions.

Two identical RC walls (W1, W2) were fabricated using a prototype 8-storey building located in Montreal, QC and scaled by a length factor l_r = 0.429. The total height of the prototype building is 20.95 m and model walls were 9.0 m high. The walls were designed according to NBCC05 and CSA-A23.3-04 for moderate ductile category (ductility-related force modification factor R_d= 2.0 and overstrengh-related force modification factor R_o=1.4) and assuming a site class C in Montreal. The uniaxial seismic simulator of Ecole Polytechnique has a payload capacity of 15 tons and 3.4 m x 3.4 m plan dimension. The 60 kN seismic masses of each floor were installed beside the table in front of each floor level on four multi-level hinged posts. The inertia loads were transferred by rigid beam that connected the wall to masses. Details of the test setup are presented in Tremblay et al. 2009 as indicated in Figure 1a, b, c.

In the test program, the two walls were subjected to several levels of a ground motion excitation spectrally matched to the NBCC 2005 design spectrum for Montreal. As it is shown in Figure 1d, 639 mm^2 to 426 mm^2 of longitudinal reinforcement bars were used along the height of the wall. As a part of the experimental program presented herein, the two wall specimens were rehabilitated after being tested and were re-subjected to the same ground motion excitations. The objective of this second phase of the study was to evaluate the effectiveness of using FRP composites for retrofitting existing RC shear walls that are susceptible to increased demand at upper floors, compared to the designed ones, due to higher mode effects. As was observed in the tests of the original walls designed according the NBCC 2005, excessive yielding of flexural reinforcement at the 6^{th} storey panel occurred, which resulted in wide horizontal cracking at the base of the 6^{th} storey. This indicates that the wall seismic demand specified by the code has exceeded the wall capacity at the 6^{th} storey level. Therefore, the rehabilitation strategy aims at increasing the flexural capacity of the wall section at the 6^{th} storey by applying vertical CFRP sheets. As a consequent of increasing the flexural capacity at that level, an increase in the shear demand would occur. Hence, the shear capacity of the wall section at the 6^{th} storey was increased as well by applying horizontal CFRP wraps. This rehabilitation scheme increases the wall strength and ductility capacity at the upper plastic hinge location. On the other hand, at the wall base, there is no need to increase the flexure capacity, which would result in an increased stiffness and, thereby, force demand of the wall. Thus, no vertical FRP strips were used at the base panel of the wall. Therefore, at the wall base the rehabilitation strategy was limited to increase the wall's ductility capacity without strength increase.

Selected Ground Motions

For the test program an ENA Mw7.0 at 70 km simulated ground motion time history was selected. Fig. 2 shows the ground motions, and the comparison between 5% damped acceleration spectrum and the Montreal NBCC 2005 target design spectrum. Wall 1 (W1) was tested under 40% (elastic), 100% and 120% of designed NBCC intensity. Wall (W2) was tested under 100%, 120%, 150% and 200% of designed NBCC intensity.

Test Results

Tables 1 and 2 show natural periods of the walls, ductility demand and rebar strains in the base and 6th floor of W1, W2. The amount of rotation ductility demand at the 6th floor for W1 and W2 are approximately equal or larger than that at the base under the application of the 100% design ground motion intensity. This behavior indicates that the walls experienced a second plastic hinge in the upper part in addition to the base hinge. The presence of the second plastic hinge is also depicted by the strain reading measured on the longitudinal rebar at the 6th floor, ε_6, which is, on average, 4 times larger than the yield strain of the bar ($\varepsilon_y = 2200\ \mu\varepsilon$).

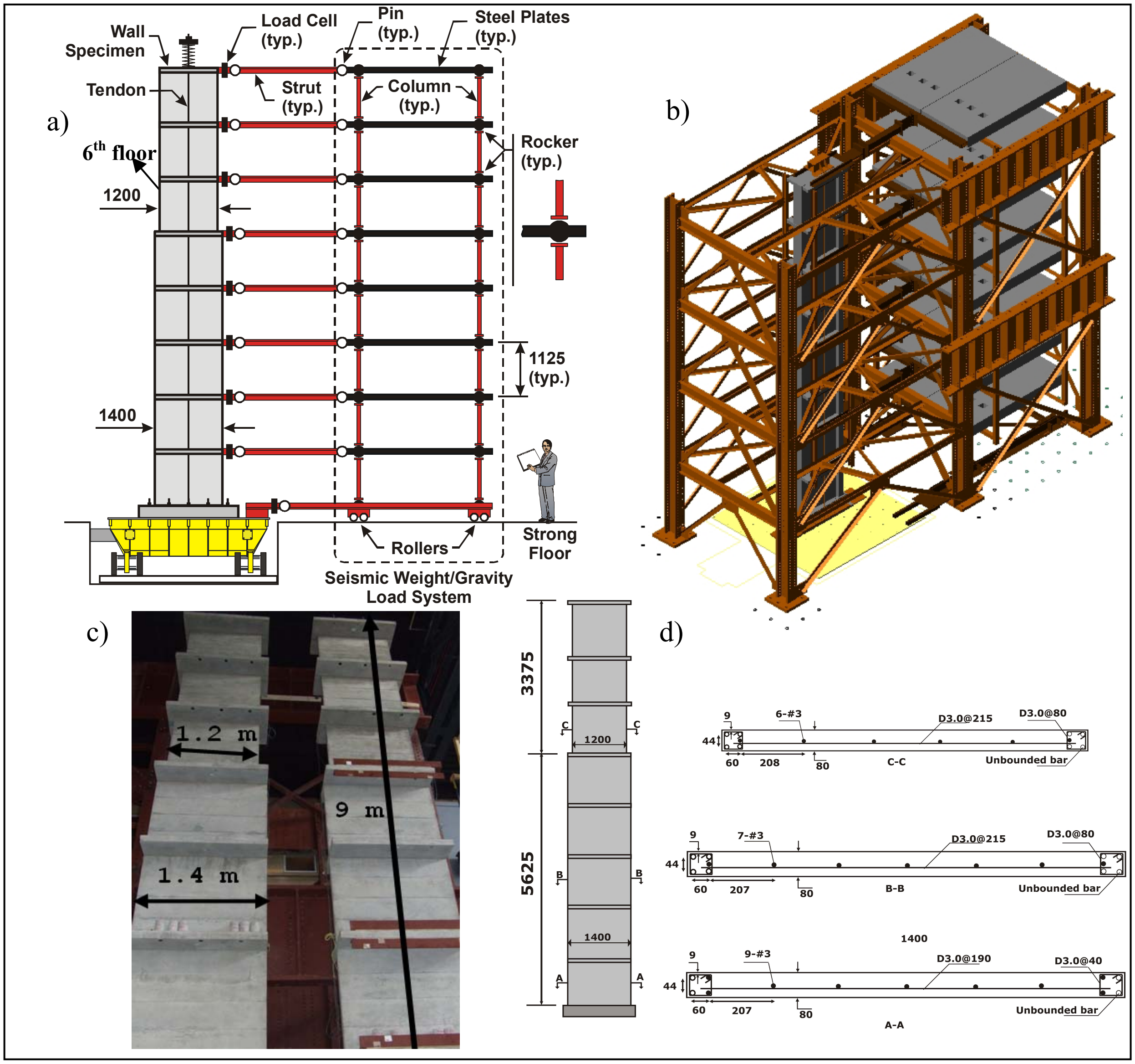

Figure 1. a) Test specimen and seismic weight/gravity load system; b) Complete test setup with stabilizing steel frame; c) model wall; and d) Cross section of model wall

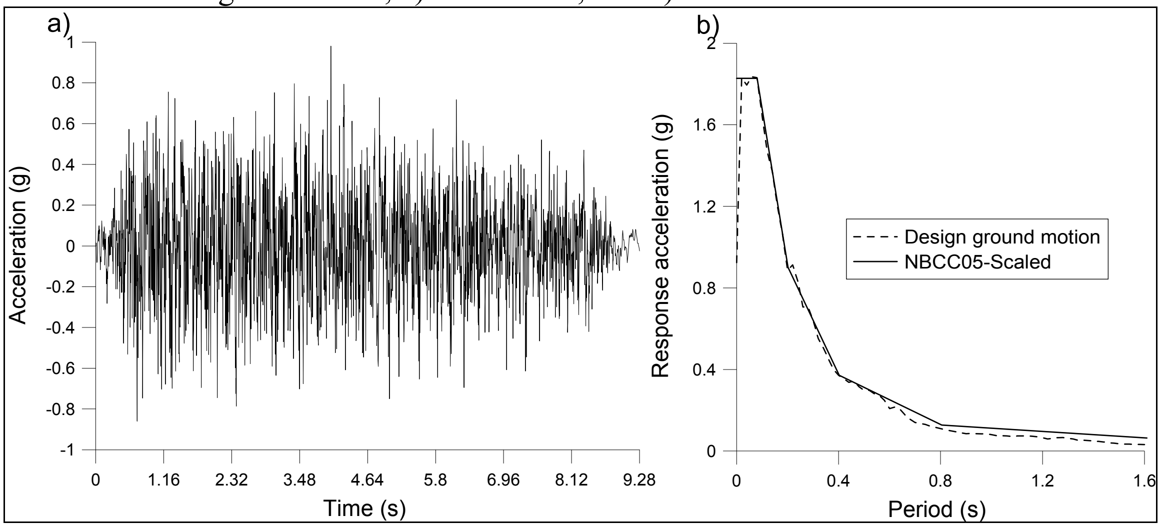

Figure 2. Selected ground acceleration: (a) time history; (b) response spectra.

Both W1 andW2 exhibited, on average, 40% elongation of their first natural period of vibration, T_1, between the initial (undamaged) condition and after the application of the designed ground motions (with 100% intensity). This represents a global damage indicator. When increasing further the ground motion intensity up to 150% for W2, no additional significant period elongation was observed and the rotation ductility demand at the base of the wall, $\mu_{\theta b}$, remained nearly unchanged. However, a significant increase in the damage and rotational ductility demand at the 6th floor was observed ($\mu_{\theta 6}$). For example, the rotational ductility demand at the base and at the 6th floor increased by 44% and 114%, respectively, when increasing the ground motion intensity from 120% to 150% for W2. Hence, damage increased at top of the wall elongating the periods associate to higher vibration modes and modifying its response accordingly.

Table 1. Key Parameters W1 – Tested first under 40% ground motion intensity (1-40%).

Test No.	**Initial**	**1 –40%**	**2- 100%**	**3 –120%**
T1 (s)	0.67	0.72	0.90	0.960
$\mu_{\theta b}$	-	0.98	4.6	5.0
$\mu_{\theta 6}$	-	1.07	6.8	9.1
ε_b ($\mu\varepsilon$)	-	1100	2350	2360
ε_6 ($\mu\varepsilon$)	-	1440	10920	9800

Figs. 3a&b show the contribution of the concrete and the horizontal reinforcement, respectively, to the resistance of the base shear for W2 under different ground motion intensities. The shear forces contributing to the horizontal reinforcement were obtained from measured steel strain during the tests and the concrete contribution was obtained by subtracting the steel contribution from the total shear forces. The values are plotted against the total base shear. The concrete shear contribution in the first quadrant of the plot in Fig. 3a has a linear variation. There is a constant steel shear contribution in the same quadrant in Fig. 3b. This means that W2 did not experience severe damage, especially in shear, in one direction. In the third quadrant of the plots in Figs. 3a&b, a reduction of the concrete contribution and an increase of the steel contribution for different ground motion intensities can be observed. This indicates that the concrete shear strength of the wall base was reduced due to an increase of rotation ductility and shear crack width.

Table 2. Key Parameters W2 – Tested first under the design ground motion (1-100%).

Test No.	Initial	1 –100%	2- 120%	3 –150%	3 –200%
T1 (s)	0.65	0.96	1.00	1.03	1.31
$\mu_{\theta b}$	-	5.2	4.8	6.9	7.1
$\mu_{\theta 6}$	-	5.6	6.6	14.1	20.9
ε_b ($\mu\varepsilon$)	-	2174	5535	11430	11368
ε_6 ($\mu\varepsilon$)	-	7118	15100	17880	-

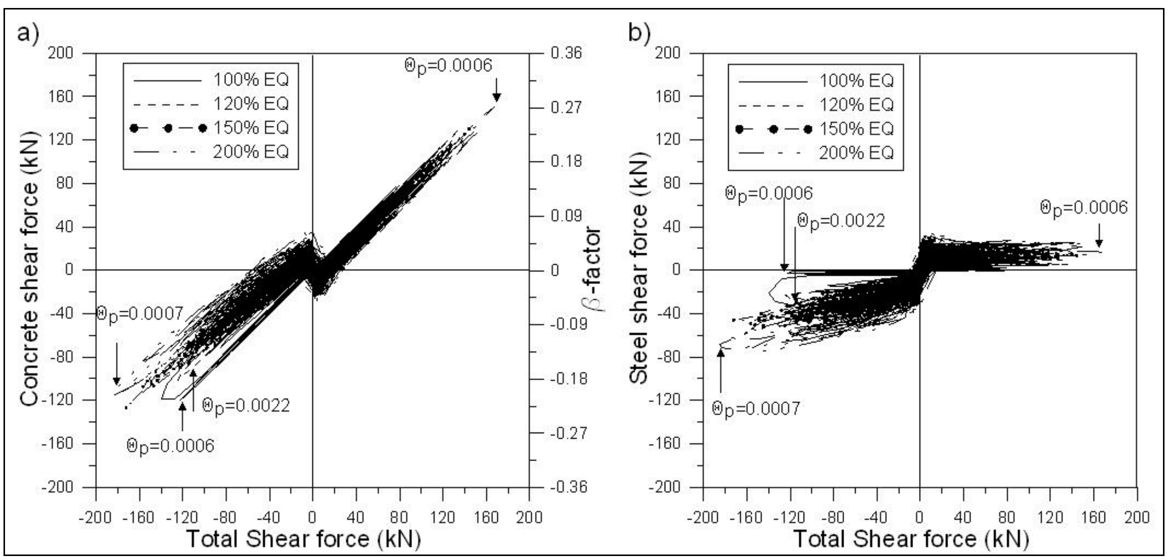

Figure 3. Shear contributions vs. total base shear (θp is the plastic rotation); (a) concrete; (b) steel.

In CSA-A23.3-04, the nominal concrete shear stress is limited to $\beta (f'_c)^{1/2}$ where β is a function of the member plastic hinge rotation (θp). Due to higher mode effects, the maximum total base shear resisted by W2 under the first test was 25% more than the nominal shear capacity predicted by the code. According to the first quadrant of the plot in Fig. 3a, this maximum total shear force occurred when the rotational ductility was limited (μ_θ=2.0). The peak value of β determined from the peak force measured in the test (0.27 in Fig.3a) is larger than the value predicted by the code (0.18), resulting in a larger contribution of the concrete to shear, which prevented yielding of the horizontal reinforcement. In the last test, where the wall experienced maximum damage, the contribution of the concrete had decreased significantly and, consequently, the stress in the horizontal steel increased up to the yield point (third quadrant in Fig. 3b). In this case the β factor, as predicted by the code, is limited to 0.18.

Seismic Strengthening of Ductile Shear Walls

FRP composite materials have been used extensively in the last few decades as a potential material for seismic retrofit of RC structures due to their high strength-to-weight ratio, high resistance to corrosion, and the ease of application. FRP laminates have been used to increase the wall flexural capacity, shear capacity, or both flexural and shear capacities by having different orientation of the laminates. Lombard et al. (2000) studied retrofitting RC shear walls using FRP composites when subjected to cyclic lateral excitations. They increased the flexural capacity, stiffness, and the shear capacity of the wall by applying one horizontal layer of CFRP sheet that is sandwiched between two vertical layers of CFRP. The vertical sheets were anchored to the foundation using steel angles. They found that FRP-retrofitted walls have better performance provided that a proper anchorage system for the sheets is used. They noted also that premature debonding of FRP sheets due to the compressive stresses in FRP vertical laminates is a critical issue in case of cyclic loading and it 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 wall shear strength and confinement. The retrofitted wall was able to reach displacement ductility levels that are 57% higher than those of the control wall, and was able to dissipate three times the energy absorbed by the original wall. Antoniades 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 FRP U-wraps horizontally at the wall end columns, and they were anchored to the wall using either steel or FRP anchors. The rehabilitated walls were able to reach high displacement ductilities compared to the control wall.

Description of FRP-rehabilitated walls (W1R, W2R)

The two original RC walls were rehabilitated and retested using the same test setup, instrumentation and under the same dynamic excitation used for the original walls. Additional strain gauges were applied on the CFRP sheets at different locations. For the first rehabilitated wall W1R, the ground motion was applied at two intensity levels; 100% and 120% of the design ground motion intensity. For the second rehabilitated wall W2R, the ground motion was applied at four intensity levels; 100, 120, 150, and 200% of the design 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 natural frequencies of the tested walls to estimate the amount of damage occurred.

As the original walls did not experience major concrete spalling, thus no concrete replacement was required. The wall surface was cleaned and grinded in several areas to achieve a smooth surface, and the wall corners were chamfered to a radius of 10 mm to avoid stress concentration upon wrapping FRP sheets. Due to the excessive yielding of the flexural reinforcement measured at the 6^{th} storey of the two original walls, the rehabilitation schemes necessitate increasing the flexure capacity at that level. For rehabilitated wall W1R, flexural capacity of the wall section at the 6th floor panel was increased by applying a 200 mm wide vertical uni-directional CFRP strip at the wall boundary zones on both sides. The vertical strips were anchored to the top and bottom slab of the 6th storey panel using FRP fan anchors as shown in Fig.4. The anchors were placed in previously drilled holes and then were filled with epoxy resins. The properties of the Tyfo SCH-11UP composites (Fyfe 2009) used in the rehabilitation scheme are shown in Table 3. In addition, the wall shear capacity at the 6th storey was increased by applying one horizontal layer of C-shaped CFRP sheet on top of the vertical strips. The C-shaped FRP sheets were overlapped at the boundary regions of the wall in order to have a better confinement of the wall end columns as shown in Figs. 5 and 6.

Figure 4. Vertical FRP strips and their FRP anchors at the 6th floor panel before applying the horizontal CFP sheets.

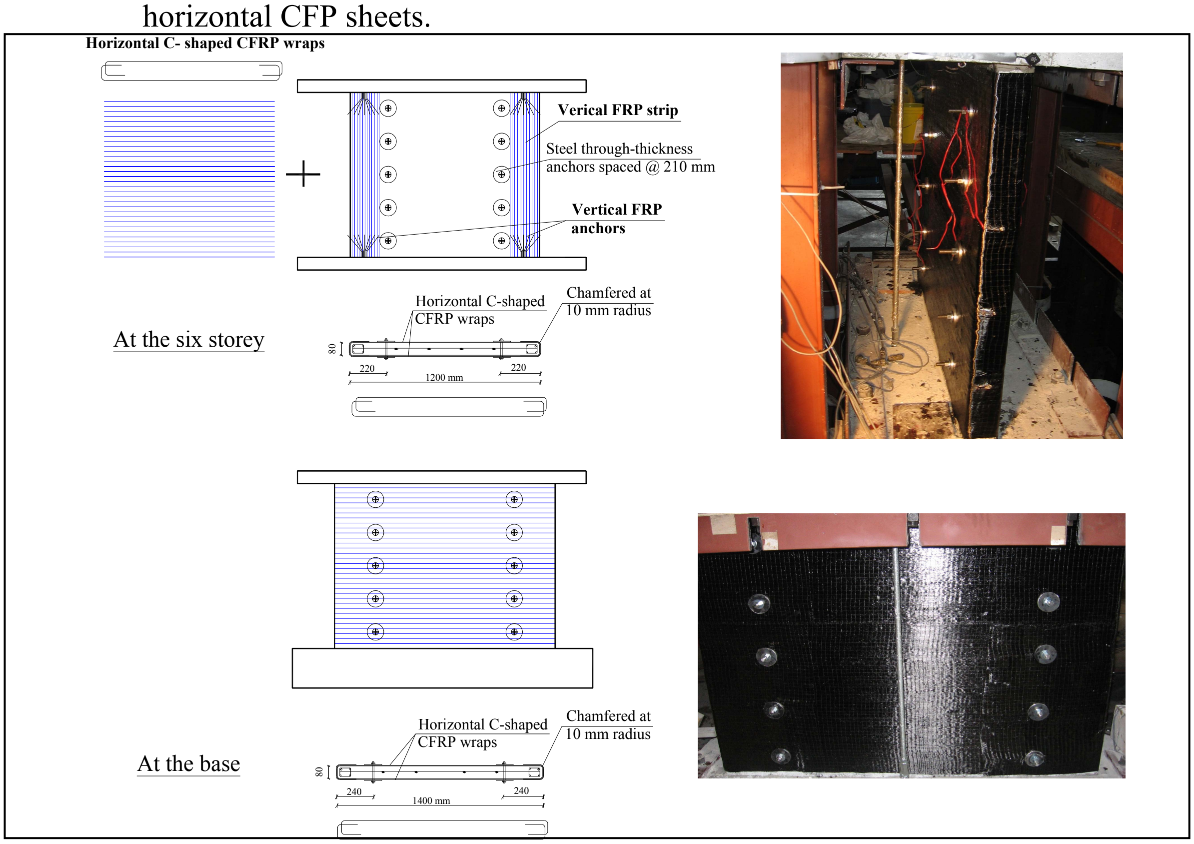

Figure 5. Details of the rehabilitation schemes for the rehabilitated walls W1R

Table 3. Properties of CFRP sheets used in the rehabilitation of the two walls (Fyfe 2009).

Properties of composite gross laminate SCH-11UP	
Composite gross laminate properties	Value
Ultimate tensile strength	903 MPa
Elongation at break	1.05%
Tensile modulus	86.9 GPa
Laminate thickness	0.27 mm

Then, the horizontal sheets were anchored along the sides of the wall using the previously drilled through-thickness steel anchors. The horizontal CFRP wraps would also prevent the premature debonding of vertical CFRP strips due to the compressive stresses. At the wall base, no increase in the flexural strength was needed. Therefore, no vertical FRP strips were used at the base storey. The panel was wrapped horizontally using the C-shaped CFRP sheets and anchored to the wall using the through-thickness steel anchors, similar to the 6th storey. Such horizontal wrapping should confine the boundary regions of the wall, thus increasing its ductility and energy dissipation capacity. For the rehabilitated wall W2R, a rehabilitation scheme similar to W1R was used for both the base and 6th stories, except that the through-thickness steel anchors were not used.

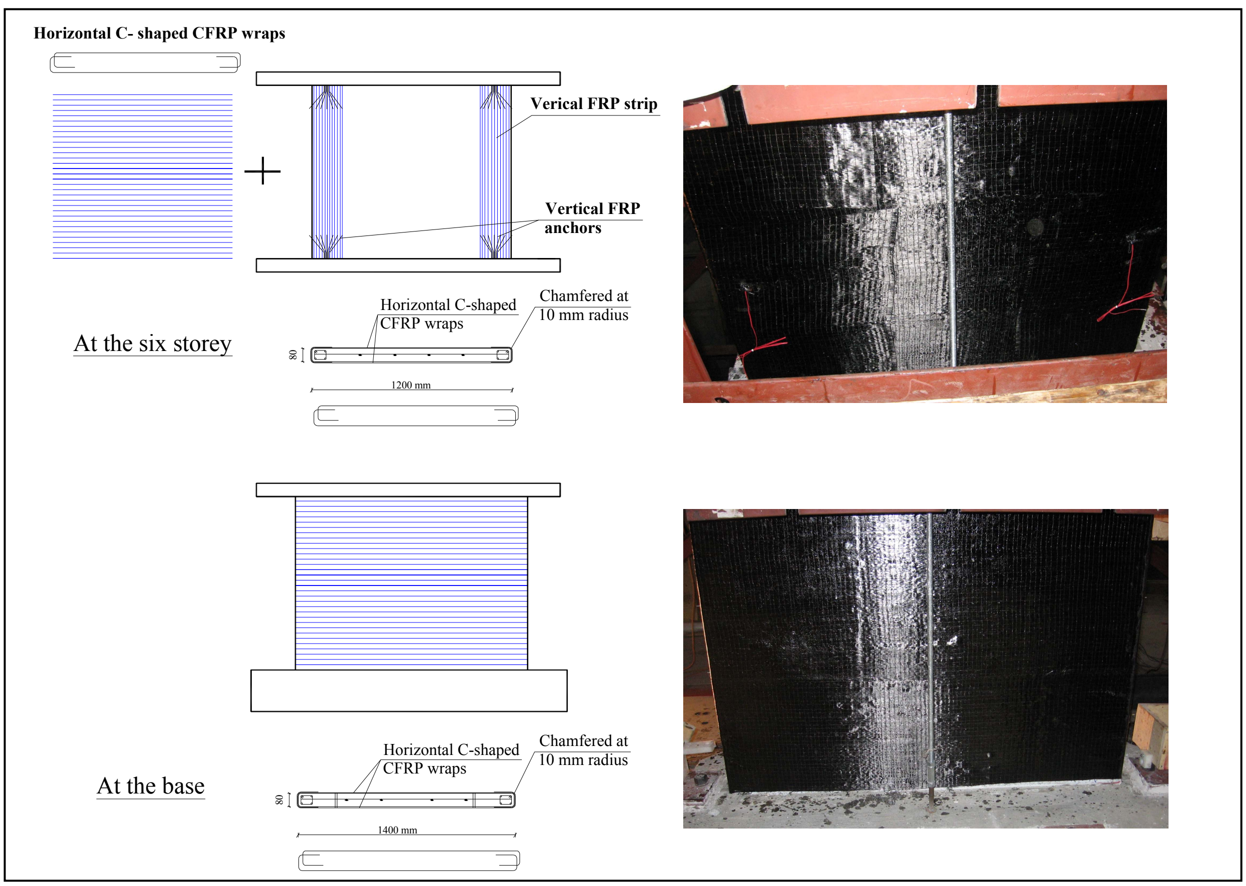

Figure 6. Details of the rehabilitation schemes for the rehabilitated walls W2R.

Test observations – Rehabilitated Walls

As was observed in the tests on the original walls, the natural frequencies of the original walls have decreased due to the accumulation of damage after each excitation. The natural frequencies of the rehabilitated walls were found to be higher than that of the original walls after being damaged, and were close to that value of the undamaged walls. The rehabilitated walls were found to perform very efficiently, no FRP debonding or anchorage failure was observed during the two tests. The vertical FRP strips applied at the 6th storey panel reduced the strains in the longitudinal steel rebars significantly at that level. In fact, the FRP was not fully utilized as the capacity of the rehabilitated walls could not be reached due to the limited capacity of the shake table. The maximum storey shear was found to be higher than that of the original walls at the same level of excitation. After applying the 120% of the design intensity on the rehabilitated wall W2R, new horizontal cracks were observed at the 2nd and 5th stories. After the 150% of the design intensity, more cracks spread in the same stories, while after the 200% of the design intensity, the cracks spread in the 3rd and 4th stories. This could be interpreted that rehabilitating the wall base and 6th storey has led to the redistribution of demands and stresses in the other unrehabilitated parts of the wall.

Conclusions

Two series of shaking table tests on 8-storey scaled model walls designed according to NBCC 2005 and CSA-A23.3-04 were carried out to investigate the higher mode effects on multistory reinforced concrete walls. Under the design earthquake, the results of the tests showed that significant rotational ductility demand occurred at the 6th storey of the wall due to higher mode effects. That demand even exceeded the base rotational ductility. Yielding of the longitudinal reinforcement at the 6th floor confirmed the significant plastic demand at this level, which resulted to a dual hinge response not accounted for in current design codes. The damage progression due to the increase of ground motion intensity is much larger in the upper part of the wall than at the base. No shear failure was observed. However, in some instances, the contribution of the concrete to shear resistance was found to be larger than the value predicted by the code.

The two original walls were rehabilitated using two different rehabilitation schemes utilizing carbon fibre-reinforced polymer (CFRP) composite sheets at the plastic hinge locations (base panel and 6th storey). The rehabilitated walls were retested by subjecting them to the same ground motion excitation levels applied on the original walls. Both rehabilitated walls performed efficiently showing improved flexural strength at the 6th storey panel. Upon increasing the seismic ground motion intensity, the damage (cracking, plasticity) was found to spread in the other unrehabilitated stories.

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