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FRP REINFORCED CONCRETE BEAMS AND COLUMNS UNDER REVERSED CYCLIC LOADING

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ABSTRACT: The use of fibre reinforced polymer (FRP) reinforcement in new concrete elements offers advantages because of non-corrosive characteristics and electromagnetic neutrality of FRP reinforcement. However, the uncertainties associated with using FRP bars in compression as longitudinal reinforcement, especially under reversed cyclic loading, as well as the brittle nature of FRPs pose challenges for their use in earthquake resistant construction. A comprehensive experimental investigation was carried out at the Structures Laboratory of the University of Ottawa to assess the performance of FRP reinforced concrete columns and beams under reversed cyclic loading. Full-scale columns, representing part of a first-storey building column between the footing and the point of inflection, and beam specimens representing beam segments between the columns and the points of inflection were tested under reversed cyclic loading. All the elements had carbon FRP (CFRP) bars as longitudinal reinforcement and CFRP grids as transverse shear/confinement reinforcement. The test parameters included the arrangement of FRP longitudinal bars, spacing of transverse FRP grids, shear span, and axial load. The behaviour was governed by the degree of concrete confinement, the level of axial compression in columns and FRP bar buckling. All the columns sustained the 2.5% seismic drift limit specified in most building codes for ordinary buildings. The hysteretic relationships indicated progressive stiffness degradation due to concrete cracking, inelasticity due to gradual concrete crushing and bar failure due to compression buckling. The beams developed approximately 2% to 3% lateral drift but showed linear behaviour until the CFRP bars ruptured in tension.

1. Introduction

Large-scale reinforced concrete columns and beams with CFRP re-bars and CFRP grids as internal longitudinal and transverse reinforcement were designed, built and tested under reversed cyclic loading to assess their performance during strong earthquakes. The experimental program consisted of 10 columns and 6 beams. The columns were first subjected axial compression of different magnitudes. The specimens were tested under incrementally increasing lateral deformation reversals. The primary objective was to assess inelastic deformability of reinforced concrete elements internally reinforced with CFRP reinforcement, without any conventional steel reinforcement, with specific emphasis on available ductility and behavior of CFRP reinforcement in compression, particularly during tension-compression cycles. Shear behavior of both columns and beams was investigated by testing elements with different shear spans and different spacing and amount of transverse reinforcement. The results of the experimental research are presented in the following sections.

2. Tests of Reinforced Concrete Columns

2.1. Properties of Test Columns

Ten cantilever columns, representing the portion of a first-storey column between the footing and the point of inflection were prepared for testing. The columns had a 355 mm square cross-section with either a 1000 mm or a 1900 mm height, which resulted in 1280 mm, or 2180 mm shear spans, respectively, up to the point of application of lateral force, after accounting for the presence of a top-loading beam. The two heights selected were used to represent shear-dominant and flexure-dominant columns. Figure 1 illustrates the geometric details of columns.

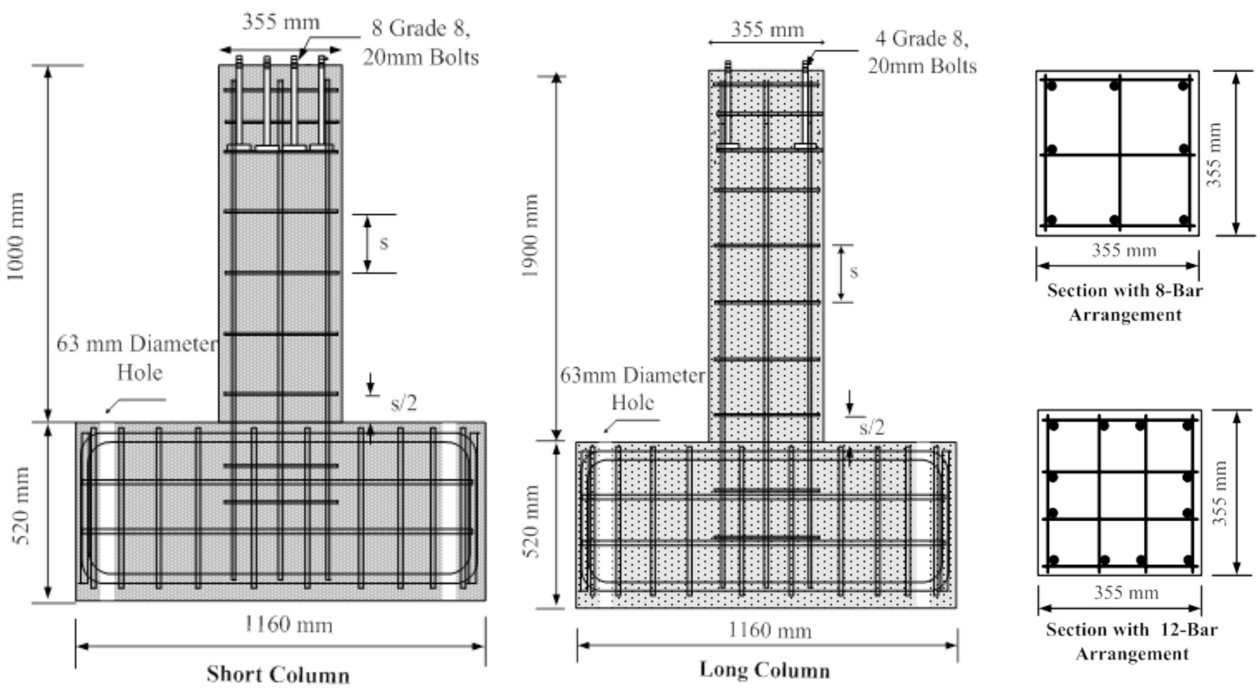


Fig. 1 – Geometric Properties of Test Columns

The longitudinal column reinforcement consisted of 9.5 mm diameter Pultrall CFRP bars. Two reinforcement ratios were used; 0.45% for the eight-bar arrangement and 0.67% for the twelve-bar arrangement. The longitudinal bars extended into the footing, which had a depth of 520 mm. The length of FRP bars inside the footing was 470 mm. Clear concrete cover was 20 mm in all cases measured from the face of the column section to the outer surface of the transverse reinforcement (NEFMAC grids). Grid spacing was either 88 mm or 175 mm. Four-cell grids were used with 8 longitudinal bars, and 9-cell grids were used with 12 longitudinal bars. Figure 2 shows the geometric properties of the two types of NEFMAC grids used as column ties.

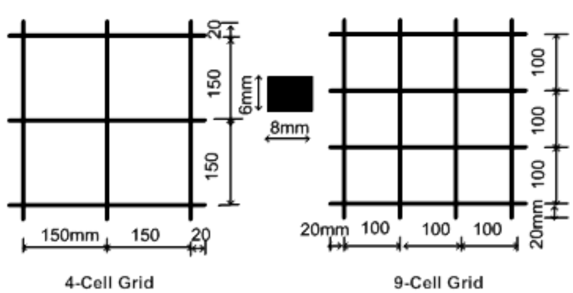


Fig. 2 – Geometric Properties of NEFMAC Grids

The test parameters included the arrangement of longitudinal reinforcement, grid spacing, shear span and the level of axial load. Table 1 provides a summary of column properties considered in the test program.

Table 1 – Properties of Column Specimens

Column	f'_c	No of 9.5 mm bars	ρ (%)	s (mm)	L (mm)	P (kN)	P/P_o (%)	Drift Push (%)	Drift Pull (%)
CFCL1	38	8	0.45	175	1900	1255	30	1.0	1.0
CFCL2	38	8	0.45	88	1900	1255	30	3.0	2.5
CFCL3	38	12	0.67	175	1900	1115	27	2.0	2.0
CFCL4	38	12	0.67	88	1900	1115	27	3.0	3.0
CFCL5	38	8	0.45	175	1000	1358	33	2.0	2.0
CFCL6	38	8	0.45	88	1000	1358	33	3.0	3.0
CFCL7	38	8	0.45	88	1000	679	17	4.0	4.0
CFCL8	38	12	0.67	175	1000	1220	30	2.0	2.0
CFCL9	38	12	0.67	88	1000	1220	30	3.0	3.0
CFCL10	38	12	0.67	88	1000	627	15	4.0	3.0

Reinforcement cages were assembled first by tying the Pultrall bars and NEFMAC grids together. The footing for each specimen was heavily reinforced with steel reinforcement to prevent premature failure in the footings. Ready-mix concrete was used to cast the footings first. The footing-column interface (construction joint) was intentionally left rough. The columns were cast vertically to simulate the actual construction practice. They were cast a few weeks later. The column cages and stages of casting are shown in Fig. 3.

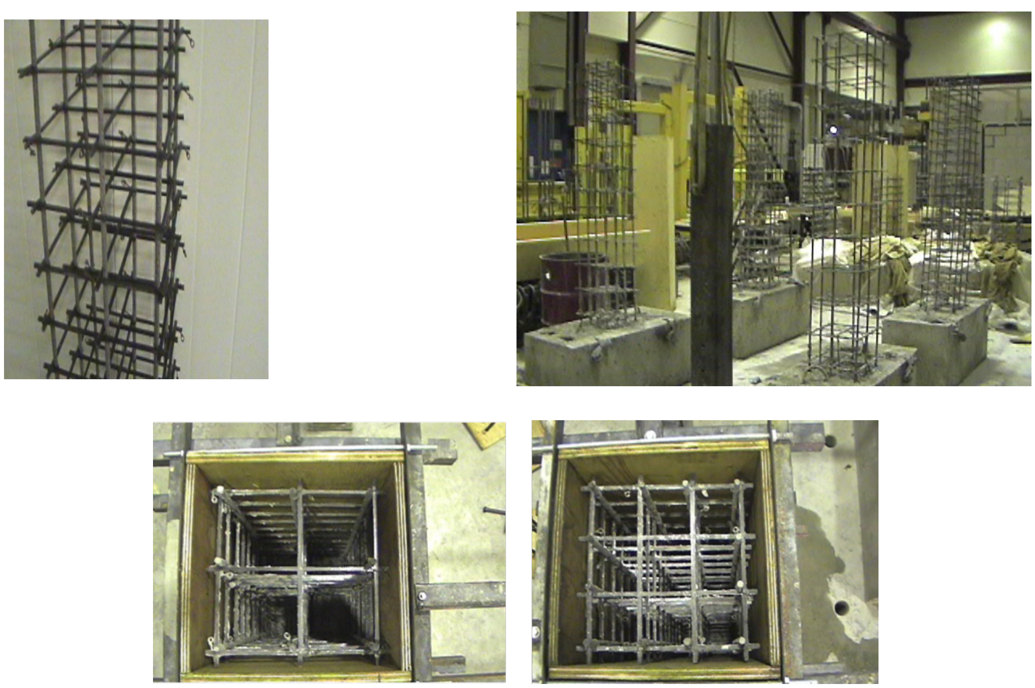


Fig. 3 – Column Cages and Stages of Concrete Casting

2.2. Test Set-up and Test Procedure

Figure 4 provides a schematic view of the test setup. A steel loading beam assembly was placed on the columns before they were connected to actuators. The specimens were instrumented with Linear Variable Differential Transducers (LVDTs), for displacement and rotation measurements. An LVDT was placed horizontally at the point of application of horizontal load to measure the column top displacement. A light aluminum frame was built around the column to attach the LVDT so that the measured displacements would be relative to the column footing. Four additional LVDTs were placed vertically near the critical section of columns to measure the rotations of hinging regions, as well as those caused by anchorage slip (extension of longitudinal reinforcement within the footing). Electric resistance strain gauges were placed on FRP bars and grids to measure strains in longitudinal and transverse reinforcement. Three 1000 kN capacity servo-controlled MTS actuators were used to apply the loads. Two of the actuators were positioned vertically to apply constant axial compression during testing. They were connected to a rigid base that had been fixed on the laboratory strong floor at one end, and to a steel-loading beam at the other end. The third actuator was positioned horizontally between the steel loading beam and the lateral support system. The maximum stroke of the horizontal actuator was 500 mm, which allowed horizontal displacement of up to ± 250 mm relative to the neutral position. The actual stroke and load during testing were monitored and recorded by two independent data acquisition systems.

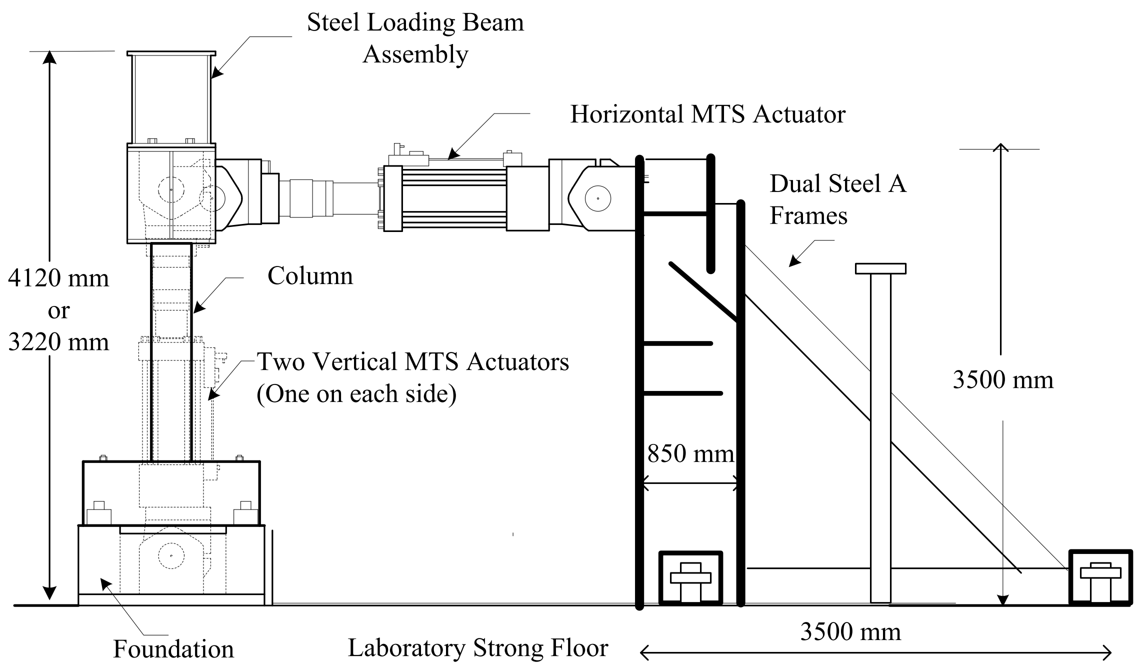


Fig. 4 – Test Set-up

The axial load was applied first and was maintained at a constant level throughout the test. The horizontal load was applied in the deformation control mode. Lateral deformation reversals were applied starting with three elastic cycles at 0.5% lateral drift, which approximately corresponded to the displacement at first flexural cracking, followed by three cycles at 1% drift. The subsequent stages of loading included three cycles at 2%, 3%, 4% etc. lateral drift until a significant drop in lateral load resistance was observed.

2.3. Material Properties

Normal Portland Cement concrete batches were ordered from a local ready-mix supplier to cast the test specimens in the Structures Laboratory of the University of Ottawa. Standard concrete cylinders were cast from each batch to establish the 28-day strength of concrete, as well as the strength during the period of testing. The actual concrete strength during the period of column tests was determined to be 38 MPa.

Pultrall CFRP bars with a nominal diameter of 9.5 mm were used as longitudinal reinforcement. The bars were made from high-strength carbon fibers and extremely durable vinyl ester resin. The carbon fibers impart strength to the bars while the vinyl ester resin imparts excellent corrosion resistance in harsh chemical and alkaline environments. The manufacturing process involved combined pultrusion and on-line coating. Bar surface was sand-coated to improve bond between the bar and the surrounding concrete. The sand coating resulted in a slight increase in bar diameter from 9.5 mm to approximately 12.0 mm. However, the latter diameter was not used in bar area and stress calculations as the sand coating did not contribute to strength.

Tensile properties of Pultrall bars were established through laboratory testing. The procedure specified in CSA Standard S806 (2012), was used as shown in Fig. 5. The coupons were 1200 mm long, with a 400 mm segment at each end placed in a steel tube to be able to grip on the coupons by the jaws of the machine during testing. The tubes had 25.5 mm inner diameter and a 3.3 mm steel thickness.

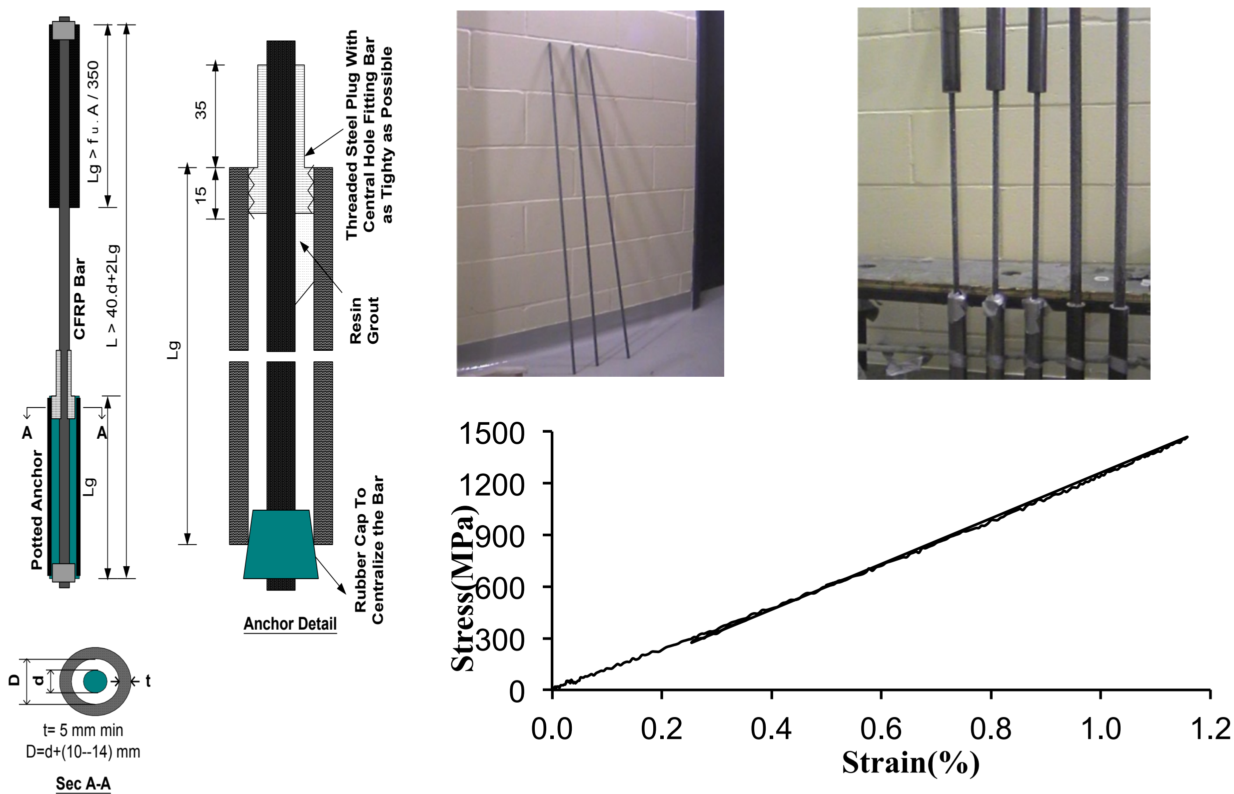


Fig. 5 – Coupon Tests of 9.5 mm Diameter Pultral CFRP Bars

The coupon tests indicated an average tensile rupturing strength of 1470 MPa and an elastic modulus of 125,000 MPa. These values are in close agreement with coupon tests reported by the University of Sherbrooke earlier during the development of the bars.

The stress-strain relationship of FRP bars in compression is difficult to establish by tests without buckling the coupons. Furthermore care must be exercised to ensure that the load is applied concentrically. Samples with lengths equal to 2 to 5 times the bar diameter were tested under direct compression until failure. The failure stress in compression varied between 240 MPa and 310 MPa. These values correspond to 16% to 21% of tensile strength. The compressive failure strain varied between 1% and 1.3%. The failure of bars in compression was caused either by delamination of fibers and crushing of resin, or by splitting of bars longitudinally. The former type of failure was observed when the compression bar sample was 3 to 5 times the bar diameter and the latter failure type was observed when the sample length was twice the bar diameter. However, the failure type did not appear to affect compressive strength and failure strain. The average modulus of elasticity in compression was found to be 23,000 MPa (about 20% of the elastic modulus in tension). The elastic modulus in compression may be thought to be representative of the modulus of resin alone, ignoring the contribution of fibers to compression. It should be recalled that Pultrall bars had vinyl ester resin with 60% fiber volume fraction.

NEFMAC grids, used as CFRP transverse reinforcement, were manufactured from carbon fibres by impregnating the fibres in vinyl ester resin. Two types of grids were used as transverse tie reinforcement; i) 6 x 8 mm rectangular bars forming four equal-size square openings, and ii) 6 x 8 mm rectangular bars forming nine equal-size square openings. They had a square configuration with 300 mm out-to-out dimension. Figure 2 and 3 illustrate the schematic view and test photographs of CFRP grids. Coupon tests were performed to establish the stress-strain relationship. The results indicated a linear relationship up to an average tensile strength of 1230 MPa, corresponding to a rupturing strain of 1.62%, with an elastic modulus of 76335 MPa.

2.4. Column Test Results

Maximum lateral drift capacities recorded during tests are listed in Table 1 for all columns. The drift capacities specified in the table correspond to maximum drift ratios prior to developing at least 20% strength decay in moment resistance, after having sustained at least two cycles of deformation at each of the previous deformation levels. The results indicate that unconfined columns tested under approximately 30% of their nominal concentric capacities could only sustain 1% to 2% lateral drift before they developed significant strength degradation due to the crushing of compression concrete followed by the buckling of compression bars. When the core concrete was confined by closely spaced transverse CFRP reinforcement, the column deformability increased to 3%. When the axial compression decreased by about 50%, the column drift capacity increased to 4%. The columns with a shorter shear span initially developed increased diagonal tension cracking. However, these columns had sufficient CFRP transverse reinforcement to supress shear failure prior to flexural failure, and were able to develop their flexural strengths.

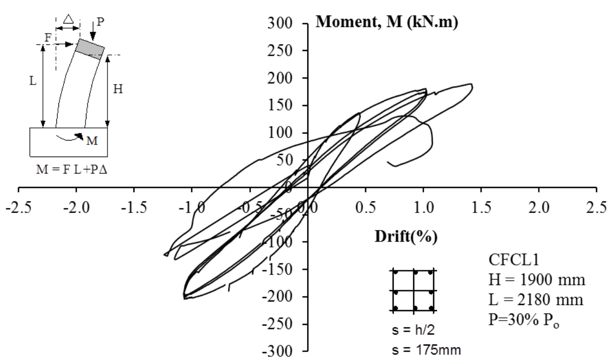
Base moment lateral drift hysteretic relationships for four columns are shown in Fig. 6. Column CFCL1 was reinforced with 8 - 9.5 mm diameter Pultrall CFRP longitudinal bars. The bars were tied with 4-cell CFRP grids with a spacing of 175 mm, which was approximately equal to twice the maximum spacing required by CSA S806-12. The column was tested under a constant compressive force of 30% of its concentric capacity. The hysteresis loops, shown in Fig 6(a) indicate stable behaviour up to 1.0% drift ratio, followed by significant and sudden strength decay due to the crushing of concrete and the stability failure of compression bars. This was expected because of wider spacing of grids, which resulted in poor concrete confinement and inadequate support for compression reinforcement. Strain gauge readings indicated that FRP bars experienced a maximum of 0.65% tensile strain and 0.50% compressive strain at the end of test, whereas the FRP grids experienced a maximum of 0.3% tensile strain.

The hysteretic relationship for unconfined column CFCL3 with 12 longitudinal reinforcement and CFRP grids with $h/2$ = 175 mm spacing is shown in Fig. 6(b). This column could sustain three cycles at 2% lateral drift, and showed significant strength degradation during the subsequent cycles at 3% drift, while also exhibiting severe concrete crushing. The longitudinal bars remained intact at this deformation level, but failed due to compression buckling at about 4% lateral drift. Strain gauge readings indicated that FRP bars experienced a maximum of 0.76% and 0.5% strains in tension and compression, respectively. The FRP grids experienced a maximum of 0.31% tensile strain.

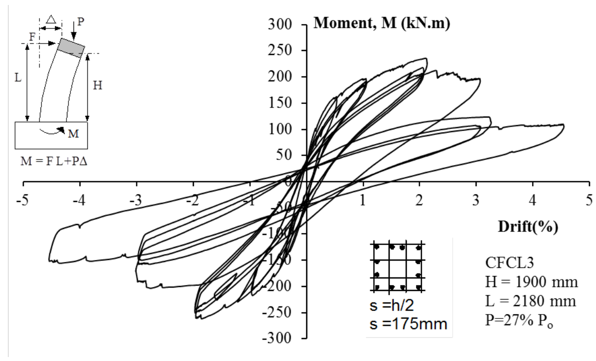
Companion column CFCL4 with 12 longitudinal bars and closely spaced ties at a spacing of $h/4$ = 88 mm showed improved behaviour. Confinement of the core concrete resulted in improved ductility and the column was able to sustain 3% lateral drift without any strength decay, but developed significant strength decay during the subsequent cycles at 4% lateral drift. It failed when forced to resist the first cycle at 5% drift ratio due to the crushing of concrete, followed by the buckling of all compression bars. Strain gauge readings indicated that FRP bars experienced a maximum of 0.93% and 0.55% strains in tension and compression at the end of testing, respectively. The FRP grids experienced a maximum of 0.5% tensile strain. Figure 6(c) illustrates the moment-displacement hysteretic relationship for Column CFCL4.

The above columns were tested under approximately 30% of column concentric capacity. Additional tests were conducted under a lower level of axial load, corresponding to 15% of column concentric capacity. Figure 6(d) shows the hysteretic relationship of CFCL10 which was tested under a lower level of axial compression. This column was a well-confined column with a 12-bar arrangement and closely spaced transverse reinforcement. It developed 4% lateral drift in push and 3% drift in pull prior to gradual strength decay. Some of the extreme compression bars failed at 4% drift ratio due to the instability of fibres. This resulted in significant strength degradation. During the last cycle at 4% drift, the cover concrete spalled off, exposing grids and broken compression bars. All of the extreme layer of FRP bars failed in compression on both sides when the column was cycles at 5% drift ratio, leaving only the middle layer of reinforcement to provide resistance. The test was stopped at this stage of loading. Strain gauge readings indicated that FRP bars experienced a maximum of 1.25% tensile strain and 0.64% compressive strain at the end of test, whereas the FRP grids experienced a maximum of 0.53% tensile strain. Though some longitudinal bars failed in tension before the end of test, the final failure of the column was triggered by the buckling of bars in compression.

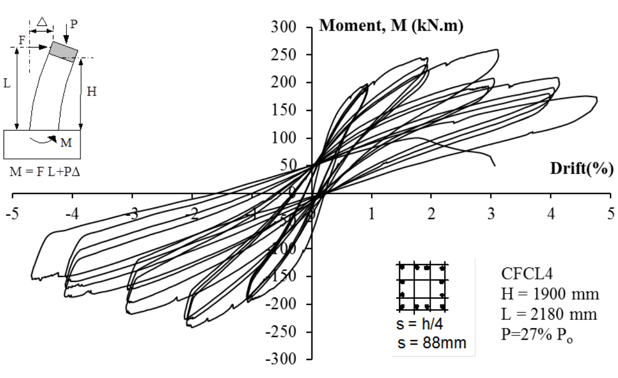
The majority of column failure was triggered by the crushing of concrete, followed by bar buckling in compression. Figure 7 illustrates typical column failure and the buckling of FRP bars in compression in the form of fibre buckling at the same location as if the bar was cut at that location while it remained straight, unlike the deformed shape typically observed in buckling steel bars.



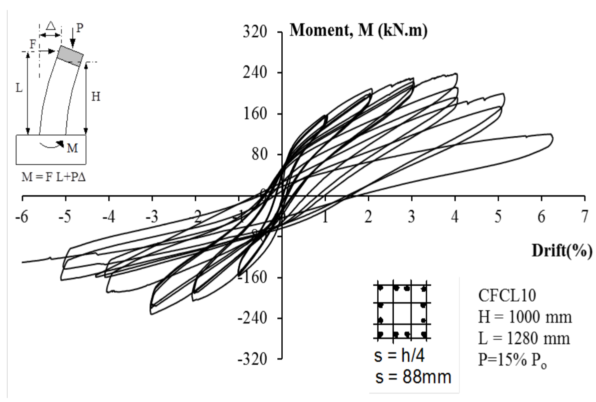
a) Column CFCL1



b) Column CFCL3



c) Column CFCL4



d) Column CFCL10

Fig. 6 – Hysteretic Moment-Lateral Drift Relationships for Columns

a) Concrete crushing

b) Buckling of FRP bars in compression

Fig. 7 – Compression Crushing of Concrete and FRP Bar Buckling in Column CFCL8

3. Tests of Reinforced Concrete Beams

3.1. Properties of Test Beams

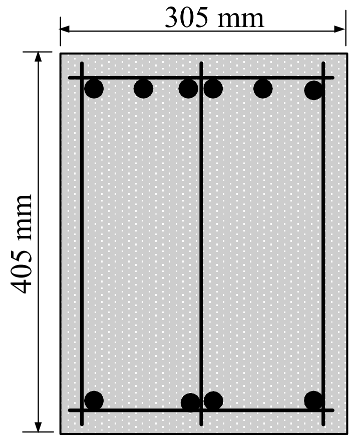
A total of six full-scale cantilever beams with a 305 mm wide 405 mm deep cross-section were tested as representatives of beam segments between the framing columns and the beam inflection points. The beams had two different lengths; either, 1000 mm or 1900 mm, corresponding to shear spans of 870 mm or 1780 respectively, measured to the point of application of shear force. The longer shear span was intended to promote flexural behaviour while the shorter span would promote shear behaviour. Four specimens were tested under reversed cyclic loading and two were tested under monotonically increasing lateral loads using the column test setup shown in Fig. 4 in vertical position without any axial load. Geometric details of specimens are shown in Figure 8.

Only one longitudinal reinforcement ratio was used in all beams. A total of 6 - 9.5 mm diameter Pultrall CFRP bars were used as top (negative) beam reinforcement and 4 same size and type bars were used as bottom (positive) reinforcement. This resulted in 0.39% top and 0.26% bottom reinforcement ratios. The longitudinal bars extended 470 mm into the footing (simulating attached column), which had a total depth of 520 mm. The clear concrete cover in all beams was 15 mm, measured from the face of the beam to the outer surface of the grid reinforcement, which was used as transverse stirrups. The grids were placed either at 180 mm spacing (corresponding to d/2) or 90 mm spacing (corresponding to d/4). They were the same as those used in columns with a rectangular configuration, having 250 x 350 mm out-to-out dimensions. Two types of grids were employed; i) 6 x 8 mm rectangular cross-section FRP reinforcement forming two equal-size rectangular openings, and ii) 8 x 10 mm rectangular cross-section FRP reinforcement forming two equal-size rectangular openings. Figure 8 illustrates the beam reinforcement and Table 2 provides a summary of beam properties and test parameters.

Normal Portland Cement concrete was used in the beams with a maximum aggregate size of 10 mm and a slump of 100 mm. The actual concrete strength during the period of beam tests was determined by standard cylinders to be 40 MPa. The properties of FRP reinforcement were the same as those discussed for column specimens in Sect. 2.3.

3.2. Beam Test Results

The hysteretic moment-lateral drift relationship for beams showed essentially elastic behavior until beam failure, with some dissipation of energy when the behaviour was dominated by flexure and the concrete was confined by closely spaced CFRP grids. However, the effectiveness of transverse grid reinforcement as confinement reinforcement was limited, and less than that for columns because of the reduced compression area due to the absence of axial load. Column CFB1 with a grid spacing of d/2 = 225 mm suffered from diagonal tension failure immediately after 2.0% drift in the strong direction. A diagonal crack occurred during the push cycles at 1% drift, which became wider during 2% drift cycles. As the diagonal cracks widened due to the insufficient transverse reinforcement the compression bars crossing the diagonal crack were subjected to dowel action. These bars eventually failed under combined flexural compression and dowel forces caused by shear. This is shown in Fig. 9. Once the longitudinal reinforcement in the compression zone was lost, the column capacity in the weak direction was lost completely as there was no tension reinforcement left in that direction. The companion column with a closer spacing of transverse grids (Column CFB2 with s = d/4 = 88 mm) sustained higher lateral forces and showed flexural behaviour. The column sustained drift cycles at 3% in the strong direction, and 2.3% in the weak direction before the CFRP longitudinal bars ruptured in tension, resulting in severe strength degradation. Beam CFB3 with the same properties as CFB2, but tested under monotonic loading showed a similar behavior, exhibiting a force deformation curve similar to the envelope of the hysteretic relationship under cyclic loading. Beams with longer shear spans showed flexural response with 3.0% to 3.5% lateral drift capacities until the longitudinal bars ruptured in flexure. Figure 10 shows typical moment-lateral drift hysteretic relationships for flexure dominant beams, either with a short shear span but sufficient transverse reinforcement or with long shear span.



Beam Cross-Section

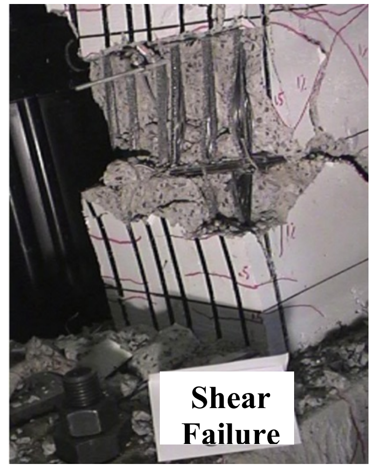
Reinforcement cage

Fig. 8 – Beam Cross-sectional Geometry and Reinforcement Details

Table 2 – Properties of Beam Specimens

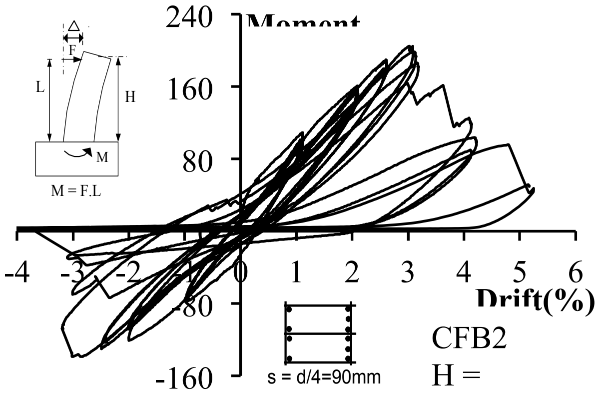
Column	Load Type	f'_c	9.5 mm top reinforcement		9.5 mm bottom reinforcement		Stirrup spacing (mm)	Shear Span (mm)	Drift Strong Axis (%)	Drift Weak Axis (%)
			No	ρ (%)	No	ρ (%)				
CFB1	Cyclic	35	6	0.39	4	0.26	175	870	2.0	2.0
CFB2	Cyclic	35	6	0.39	4	0.26	88	870	3.0	2.3
CFB3	Monotonic	35	6	0.39	4	0.26	88	870	3.0	--
CFB4	Cyclic	35	6	0.39	4	0.26	175	1780	3.5	3.0
CFB5	Cyclic	35	6	0.39	4	0.26	88	1780	3.0	3.0
CFB6	Monotonic	35	6	0.39	4	0.26	88	1780	3.5	--

a) Immediately after shear failure

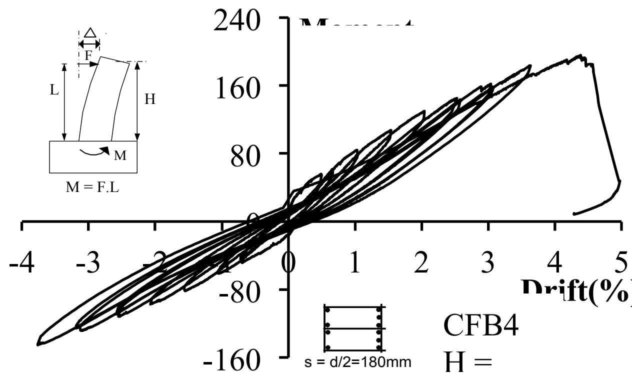


b) At the end of test

Fig. 9 – Diagonal tension failure in Beam CFB1



a) Beam CFB2



b) Beam CFB4

Fig. 10 – Typical Hysteretic Relationships for Flexure-dominant Beams

4. Conclusions

The following conclusions can be drawn from the experimental research reported in this paper:

- FRP reinforced concrete columns show improved hysteretic behaviour if confined with closely spaced CFRP grids. The drift capacities observed in the current investigation under reversed cyclic deformations indicated 1% to 2% for poorly confined columns and 3% for well-confined columns when subjected to about 30% of their concentric capacity. The column drift capacity increased up to 4% lateral drift when the axial load level was reduced in well-confined columns to about 15% of column concentric capacity.
- CFRP bars can be used as column longitudinal reinforcement for columns subjected to reversed cyclic loading. The flexural failure of such columns is triggered by the crushing of compression concrete, followed by the failure of compression reinforcement due to fibre buckling. The FRP longitudinal reinforcement performed well under cyclic loading up to 0.5% to 0.6% compressive strains, suggesting that there is a limit to the improvement in concrete behaviour that can be relied on due to confinement before the compression bars experience failure. The coupon tests of the CFRP bars used in the current investigation showed that the strength in compression was about 16% to 21% of their strength in tension, and the elastic modulus in compression was about 20% of that obtained in tension.

- Flexure-dominant CFRP reinforced concrete beams exhibit virtually linear behaviour under reversed cyclic loading, with reduced elastic stiffness, developing flexural failure at approximately 3% lateral drift due to the rupturing of tension bars. Beams with insufficient shear reinforcement and/or reduced shear span may develop diagonal tension failure at reduced drift levels. A beam tested in the current investigation, with a stirrup spacing of d/2 and a shear span to depth ratio of 2.15 developed diagonal tension failure at about 2% lateral drift.
- FRP reinforced concrete beams and columns in seismically active regions should be designed with care, with due considerations given to their hysteretic response. While these structures have longer periods as compared to conventional construction with steel reinforced concrete elements, and hence may attract lower seismic forces, hysteretic energy dissipation in such members are limited to well-confined columns. It is possible to attain 3% lateral drift in well-designed CFRP reinforced concrete structures. However subsequent failure may be brittle, triggered by either the rupturing of tension reinforcement as typically observed in beams or the compression failure of bars as typically observed in columns, unless appropriate material resistance factors are applied to ensure elastic behaviour of FRP during seismic response.

5. References

CSA (2012). Design and construction of building structures with fibre-reinforced polymers Seismic Evaluation and Retrofit of Existing Buildings (CSA S806-12). *Canadian Standards Association,* Mississauga*, Ontario, Canada*, 184 p.