13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 2717

BEHAVIOUR OF FRP REINFORCED CONCRETE UNDER SIMULATED SEISMIC LOADING

M. Kazem SHARBATDAR[1] and Murat SAATCIOGLU[2]

SUMMARY

Characteristics of concrete columns and beams reinforced with fiber reinforced polymer (FRP) reinforcement were examined under simulated seismic loading. Specimens representing part of a first storey building column between a footing and the point of inflection; and the portion of a beam between a column and the point of inflection, were tested under lateral deformation reversals. The members were reinforced with carbon FRP bars in the longitudinal direction, and carbon FRP grids in the transverse direction. Both the columns and the beams sustained a minimum of 2% to 3% lateral drift, meeting seismic drift limitations of most building codes. Hysteretic relationships indicate progressive stiffness degradation due to concrete cracking, followed by inelasticity in columns due to gradual crushing of confined concrete. They further indicate softened response of beams within the elastic range, caused by reduced crack control associated with lower modulus of FRP reinforcement, followed by the tensile rupturing of bars. The shape of hysteresis loops was distinctly different than that of steel reinforced concrete elements. The columns developed yield-like behavior caused by gradual crushing of concrete, followed by a plateau region sustained by confined concrete and unloading branches passing through the origin, resulting in reduced energy dissipation while sustaining significant inelastic lateral drift. The beam response was limited to elastic behaviour, though significantly softened due to progressive cracking, followed by the rupturing of FRP bars in tension. The results further indicate that FRP reinforced concrete structures may have the required strength and deformability expected of earthquake resistant structures, where deformability is provided partly by concrete confinement and partly by the deformability of cracked concrete elements reinforced with low modulus FRP reinforcement.

INTRODUCTION

Fiber reinforced polymer (FRP) reinforcement, in the form of longitudinal and transverse reinforcement, are currently being developed for use in new buildings and bridges. The major driving force behind this development is the superior performance of FRPs in corrosive environments. FRP reinforcement has high strength-to-weight ratio, favorable fatigue strength, electro-magnetic transparency and low relaxation characteristics when compared with steel reinforcement, offering a structurally sound alternative in most

---

[1] Assistant Professor, Department of Civil Engineering, Semnan University, Semnan, Iran. Formerly, PhD student in the Department of Civil Engineering of the University of Ottawa, Ottawa, Canada, K1N 6N5.

[2] Professor and University Research Chair, Dept. of Civil Engineering, Univ. of Ottawa, Ottawa, Canada, K1N 6N5, E-mail: murat@eng.uottawa.ca

applications. However, FRP reinforcement shows linear stress-strain characteristics up to failure, without any ductility. This poses serious concerns about their applicability to earthquake resistant structures, where seismic energy is expected to be dissipated by inelasticity in members.

Experimental research has been underway at the Structures Laboratory of the University of Ottawa to investigate seismic performance of FRP reinforced concrete structural elements. Large scale columns and beams have been tested under simulated seismic loading. The results of selected tests are summarized in the following sections, with the assessment of their significance from seismic performance perspective.

EXPERIMENTAL PROGRAM

Properties of Test Specimens

The experimental program consisted of two types of reinforced concrete elements; i) square columns, and ii) rectangular beams. The specimens were reinforced with carbon FRP bars and carbon FRP grids as longitudinal and transverse reinforcement, respectively. They represented portions of column and beam elements between rigidly attached adjoining members and the points of contraflexure, as cantilever specimens.

Columns

The columns had a 355 mm square cross section with a 1900 mm height which resulted in a 2180 mm shear span when measured form the point of application of lateral load. Figure 1 illustrates the geometric details of specimens. The columns were reinforced with 12-9.5 mm diameter carbon FRP bars, resulting in 0.7 % longitudinal reinforcement ratio. The bars continued into the footing by 470 mm, which had a total depth of 520 mm. Carbon fibre grids were used as column ties. The grids had nine cells and were manufactured from 6 x 8 mm FRP bars with overlapping fibers at intersecting joints. They had a square configuration with out-to-out dimension of 300 mm. One longitudinal bar was placed in each perimeter corner. The grid spacing was either 88 mm or 175 mm. Table 1 provides a summary of column properties considered in the test program.

Beams

The beams had 305 mm width and 405 mm depth. They had a cantilever length of 1900 mm, and a shear span of 1780 mm, measured from the point of application of load. The beams were reinforced asymmetrically to simulate the actual arrangement used in practice. Accordingly, top and bottom reinforcement consisted of 6 and 4 - 9.5 mm diameter carbon FRP bars, respectively, resulting in 0.39% and 0.26% tension reinforcement ratios in strong and weak directions, respectively. Carbon fibre grids were used as stirrups. The grids had two cells and were manufactured using 6 x 8 mm rectangular FRP bars with overlapping fibers at intersecting joints. The perimeter dimensions of grids were 250 x 350 mm and the resulting total area of transverse reinforcement effective against shear (in the direction of loading) was 144 mm^2. The grid spacing was either 90 mm or 180 mm. Figure 2 and Table 1 provide the details of beam properties.

Material Properties

Concrete

Two different batches of Normal Portland Cement concrete were used to cast the columns and beams. Separate batch of concrete was used to cast the footings for specimens, through which they were secured to the laboratory strong floor. Concrete strength gain was monitored by performing standard cylinder tests periodically. The concrete strengths during the time of testing were 37 MPa and 40 MPa for column and beams, respectively.

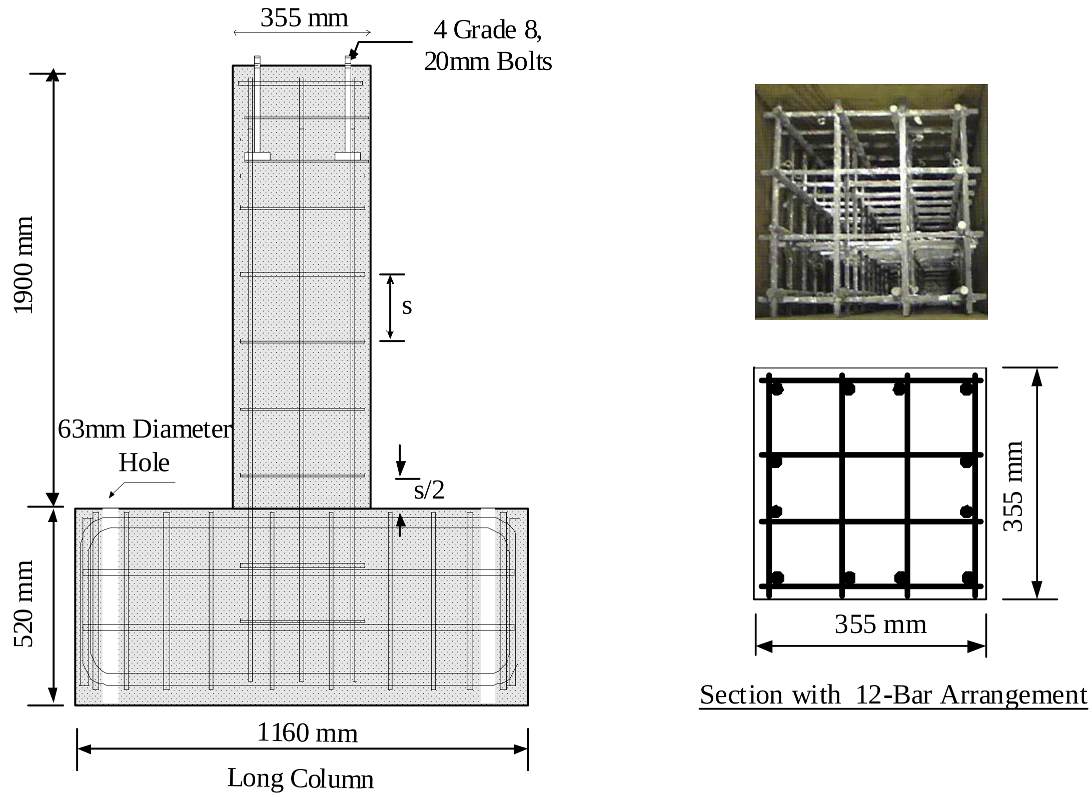


Figure 1 Details of column specimen

Table 1 Properties of test specimens

	Specimen	f'_c (MPa)	Reinforcement Arrangement	ρ (%)	S (mm)	L (mm)	P (kN)	P/P_0 (%)
Columns	CFCL3	37	12-9.5 mm bars	0.7	175	1900	1115	27
	CFCL4	37	12-9.5 mm bars	0.7	88	1900	1115	27
Beams	CFB4	40	6-9.5 mm bars (+ve) 4-9.5 mm bars (-ve)	0.39 0.26	180	1900	0	0
	CFB5	40	6-9.5 mm bars (+ve) 4-9.5 mm bars (-ve)	0.39 0.26	90	1900	0	0

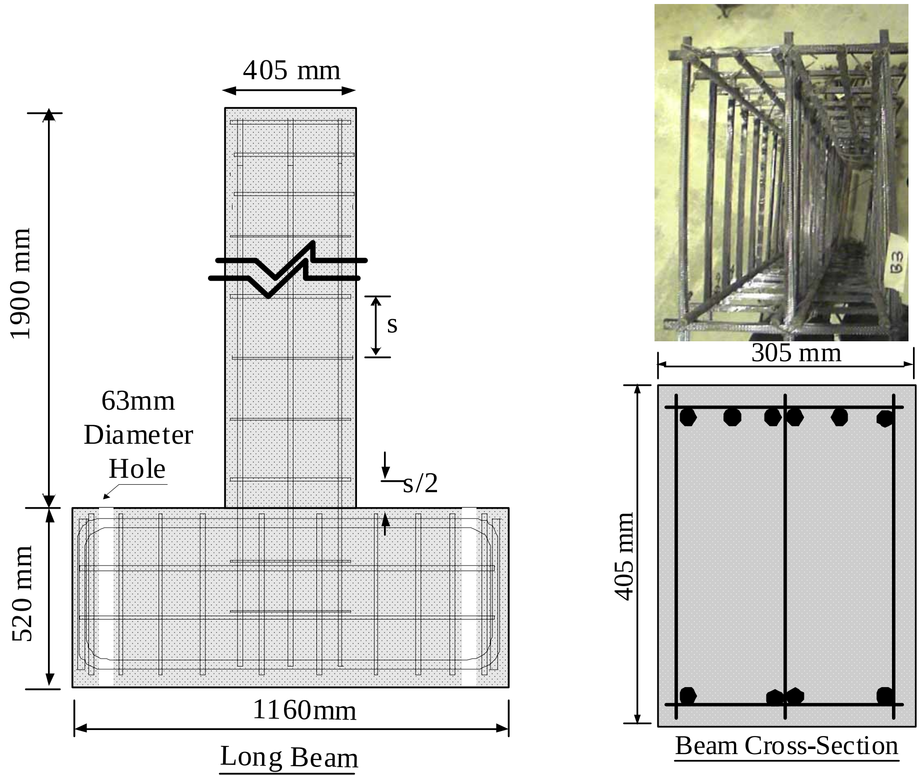


Figure 2 Details of beam specimens

FRP Reinforcement

The longitudinal reinforcement was manufactured by Pultrall Inc. with a nominal diameter of 9.5 mm. They were made from high strength carbon fibers and extremely durable vinyl ester resin. The bar surface was sand-coated for improved bond. Sand coating increased the bar diameter to approximately 12 mm. Coupon tests were conducted to establish the stress-strain relationship in tension. Figure 3 illustrates the experimentally obtained stress-stress relationship. The average tensile strength and modulus of elasticity were 1450 MPa and 122,000 MPa, respectively.

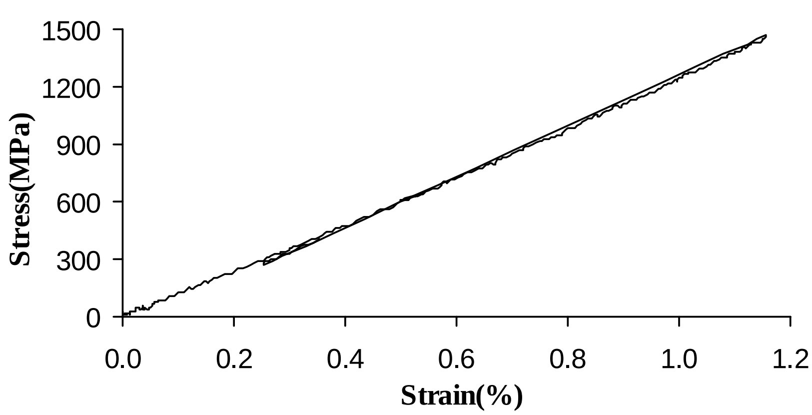


Figure 3 Stress-strain relationship of Pultral FRP bars

The stress-strain relationship of FRP bars in compression was difficult to establish by tests because of the possibility of encountering stability failure. Therefore, short samples having lengths equal to 2 to 5 times the bar diameter were tested under direct compression until failure. The average modulus of elasticity in compression was 23,000 MPa, which was approximately 20% of the value in tension. The failure stress in compression varied between 240 MPa and 310 MPa. These values correspond to 16 % to 21 % of tensile strength. The failure strain in compression varied between 1% and 1.3%. The failure in direct compression was caused either by delamination of fibers and crushing of resin or by splitting of bars longitudinally.

NEFMAC^{TM} grids, used as transverse reinforcement is manufactured by forming flat or curved grids through a pin-winding process, similar to filament winding. The product used in the current phase of experimental research was reported to have a specific gravity of 1.4 t/m^3 and a modulus of elasticity of 100,000 MPa, by the manufacturer. The grids used as column ties had a square configuration with 300 mm out-to-out dimension. These grids were manufactured using cross FRP bars, each having 6 x 8 mm rectangular cross-section, forming nine equal-size square openings. The grids used as beam stirrups had two rectangular cells, simulating perimeter hoop and a vertical crosstie. The stress-strain relationships obtained from coupon tests are illustrated in Figure 4 and indicate maximum tensile strength of 1230 MPa and elastic modulus of 76,335 MPa.

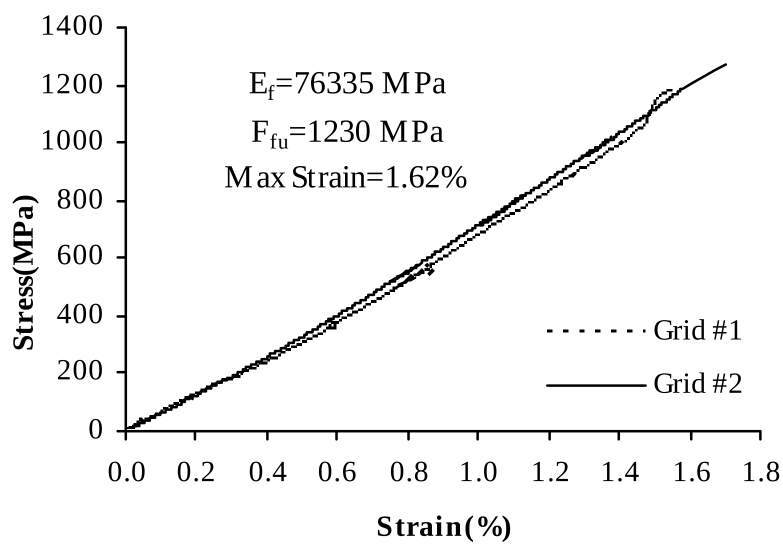


Figure 4 Stress-strain relationships for NEFMAC grids

Preparation, Test Setup and Instrumentation

Reinforcement cages were assembled first, by tying the Pultrall bars and NEFMAC grids together. Column specimens consisted of a footing and a cantilevering portion of a column. Beam specimens consisted of the segment of an attached column and a cantilevering portion of a beam. The attached portion of column was assumed to be rigid, providing full fixity at the end of the beam. Both beam and column specimens were prepared and tested vertically using the same test setup. The footings for the column specimens and the attached portions of columns for the beam specimens were heavily reinforced with steel reinforcement. Plywood formwork was prepared separately for casting each specimen in two stages to simulate the actual process of casting in practice. First the attached members were cast (footings for column specimens and column segments for beam specimens) as illustrates in Figure 5. The columns and beams were then cast vertically a few weeks later.

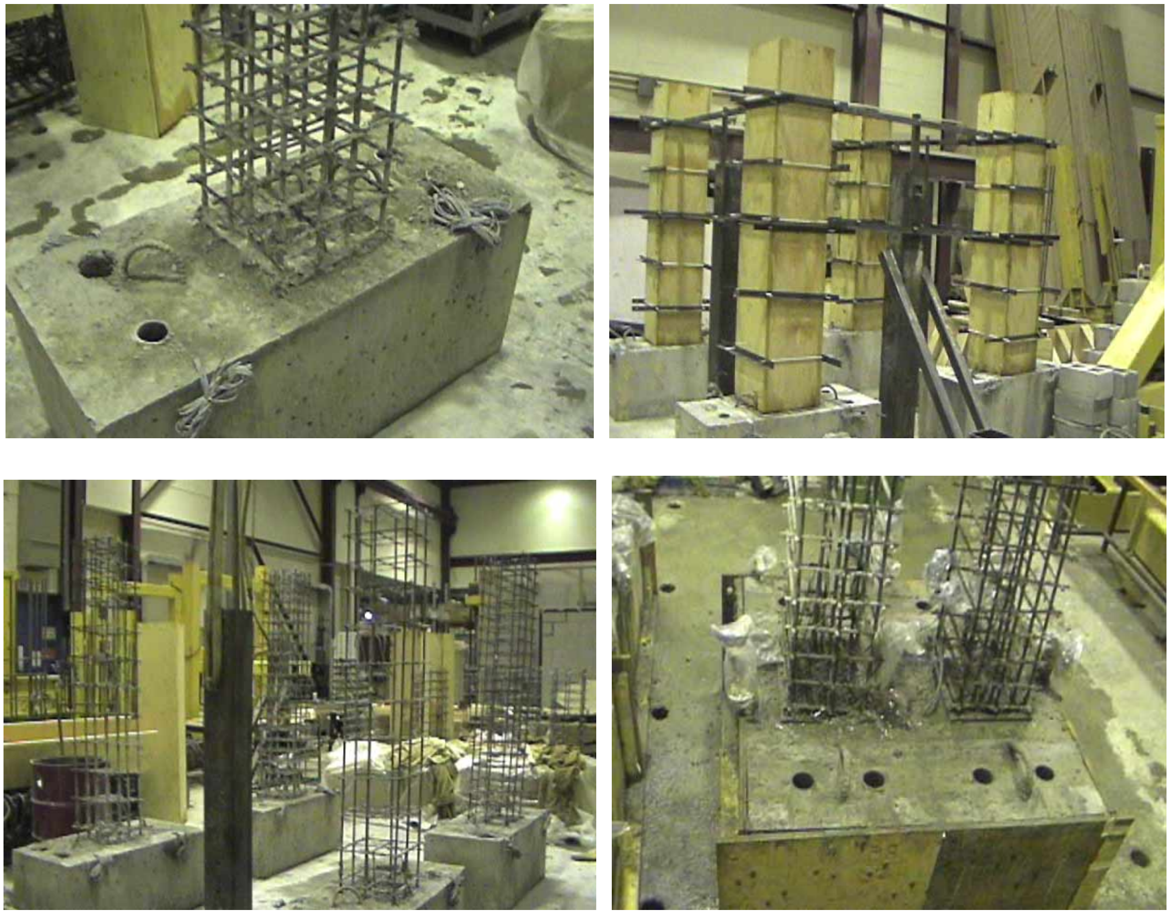
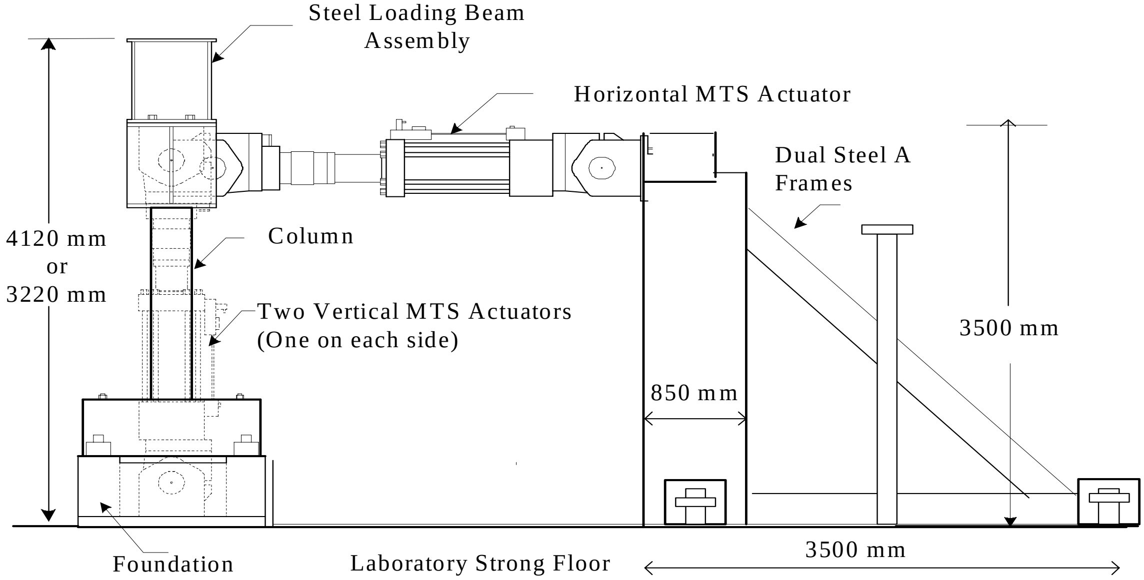


Figure 5 Views of column and beam cages and the two stage casting process employed

Once gained sufficient strength, the specimens were mounted on a heavily reinforced I-shaped foundation, which was secured on the laboratory strong floor and allowed for the assembly of vertical actuators on either side. Steel loading beam assembly was secured on the columns before they were connected to actuators. Four-Grade 800 MPa, 20 mm bolts had been cast in column specimens and were used for this purpose. Figure 6 illustrates the test setup for column tests. The beams were tested by the same test setup, except the lateral load was applied directly on the specimens at their ends through steel plates, as illustrates in Figure 7. 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 on columns 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 beams were only subjected to lateral deformation reversals and hence the vertical actuators were not used. The maximum stroke of the horizontal actuator was 500 mm, which allowed horizontal displacement of up to $\pm$ 250 mm relative to the neutral position. The actual stroke and load during testing were monitored and recorded by two independent data acquisition systems.

The specimens were instrumented with Linear Variable Differential Transducers (LVDTs), for displacement and rotation measurements. A Temposonic LVDT was placed horizontally at the point of application of horizontal load to measure the specimen tip displacement. A light aluminum frame was built around the specimens to attach the Tempononic LVDT so that the measured displacements would be relative to the attached concrete member (footings for column specimens and columns for beam specimens).



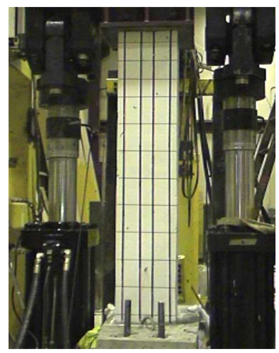


Figure 6 Test setup

Four additional LVDTs were placed vertically near the critical section of columns and beams to measure the rotations of hinging region, as well as those caused by anchorage slip (extension of longitudinal reinforcement within the adjoining element). These LVDTs were placed in pairs, one on each side, perpendicular to the direction of loading. Figure 7 illustrates the locations of LVDTs on a typical test specimen. Electric resistance strain gauges were placed on FRP bars and grids to measure strains in longitudinal and transverse reinforcement. All the instrumentation, including load cells and stroke LVDTs of MTS actuators were connected to data acquisition systems and MTS controller for data collection. Two microcomputers were used to control the data acquisition systems and the MTS controller.

Test Procedure

The specimens were first fixed on the laboratory strong floor by means of steel bolts. They had full fixity at their supports. Both the columns and beams were tested in vertical position to accommodate the same test setup. The columns were first loaded to 27% of their concentric capacities to simulate gravity loading. This load level of 1115 kN remained constant throughout the test. Horizontal loading is applied using the actuator in its deformation control mode. Lateral displacements were applied in reversed cyclic mode, three cycles at each deformation level. Following initial three cycles at approximately 0.5% drift, subsequent displacement reversals were applied in increments of 1% until a significant strength drop was observed. The beams were tested similar to the columns, except they were not loaded with initial axial compression.

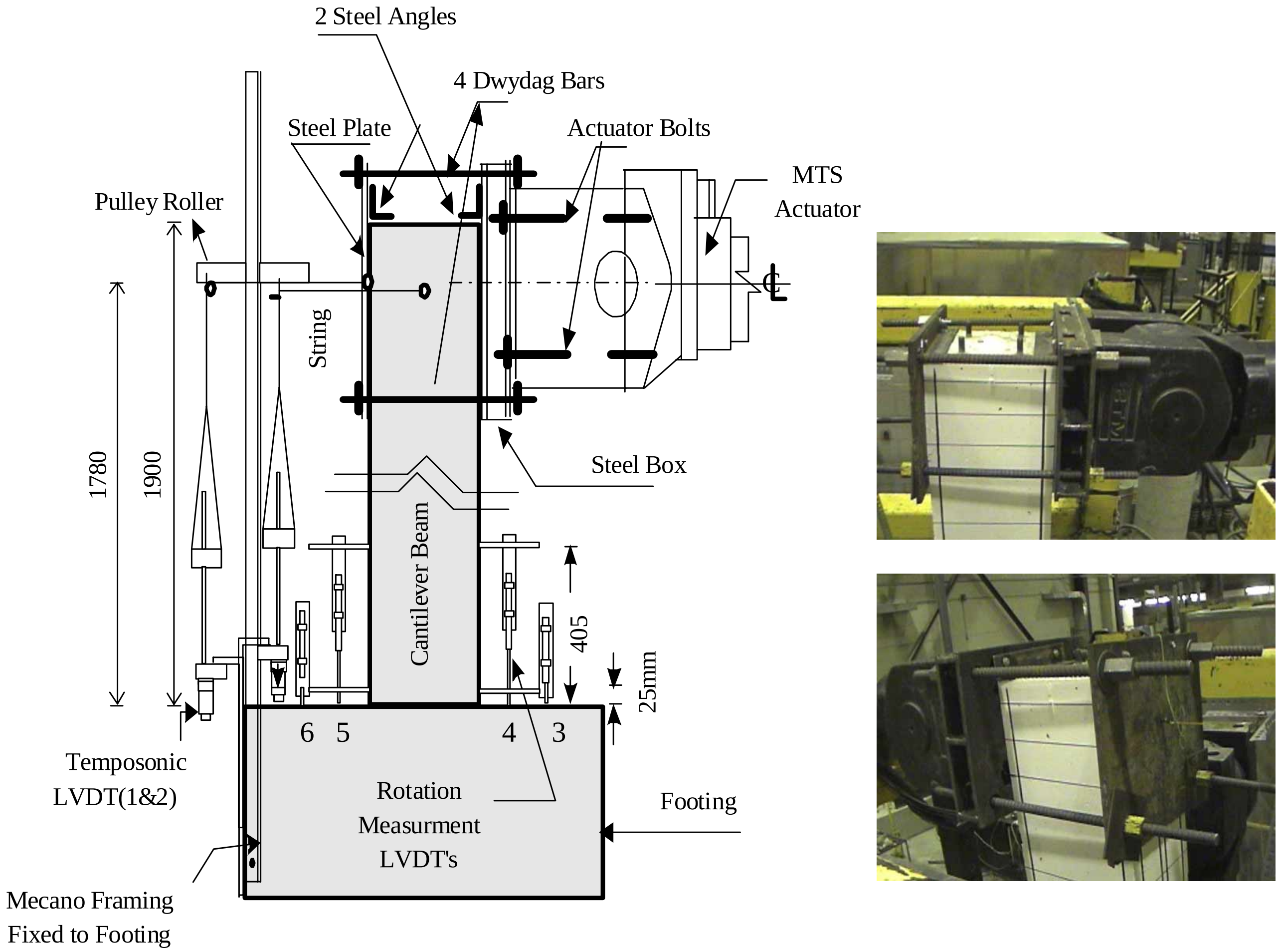


Figure 7 Application of lateral load on beams and typical instrumentation

Design of Test Specimens

Flexural design of columns was done by constructing moment-axial force interaction diagrams for columns. The interaction diagrams for FRP reinforced sections are different than those established for steel reinforced columns. The concept of "balanced section" has a different meaning for FRP reinforced sections since FRP reinforcement does not yield. The balanced load in interaction diagrams also changes its meaning and does not necessarily correspond to the maximum moment resistance point. The columns were designed to sustain 27% of their concentric capacity, as expected in some building columns. A relatively low percentage of reinforcement of 0.7 % was used as compared to steel reinforced columns, because of the higher strength of FRP reinforcement. The effects of FRP reinforcement in compression were incorporated in design with due considerations given to their stress-strain characteristics established experimentally and discussed later in the paper.

An important aspect of design was concrete confinement since column deformability could only be introduced through concrete confinement due to the brittle behaviour FRP bars. Carbon FRP grids were used as column confinement reinforcement. The displacement based design approach developed by

Saatcioglu and Razvi [1] and also adopted by the Canadian Standard CSA S806-02 [2] was used for confinement design. Accordingly, the required area of transverse FRP grid reinforcement was calculated as specified below:

$$A_{Fh} = 14 s h_c \frac{f'_c}{f_{Fh}} \left(\frac{A_g}{A_c} - 1\right) \frac{\delta}{\sqrt{k_c}} \frac{P_f}{P_{ro}} \qquad (1)$$

$$\left(\frac{A_g}{A_c} - 1\right) \geq 0.3$$

Where;

$$\frac{P_f}{P_{ro}} \geq 0.2$$

$$k_c = 0.15 \sqrt{\frac{h_c}{s} \frac{h_c}{s_\ell}}$$

A_c : Core area of column core measured to the centerline of grid perimeter.
A_{Fh}: Area of transverse FRP reinforcement perpendicular to core dimension h_c.
A_g : Gross cross-sectional area of column.
E_F : Elastic modulus of FRP grids in tension.
f'_c : Specified compressive strength of concrete
f_{Fh} : 0.004 E_F
h_c : Cross-sectional dimension of column core perpendicular to the direction of lateral load.
P_f : Factored design axial loads with zero end eccentricity (applied concentric load).
P_{ro} : Factored load resistance under zero end eccentricity (concentric load resistance).
s : Spacing of transverse grids.
s_l : Spacing of grid legs in column cross-sectional plane, perpendicular to core dimension h_c.
δ : Design lateral drift ratio, shall not be less than 3%.

The maximum spacing of FRP grids, permitted by CSA S806-02 [2] is the lesser of one-quarter of member dimension or 150 mm. In addition, a restriction on spacing is placed as 6 times the diameter of longitudinal bars. However, the latter requirement is intended for the stability of steel reinforcement since the current CSA S806-02 does not permit the use of FRP bars in columns due to lack of research in the area. The FRP bars typically have smaller cross-sections than steel bars because of their higher strength. The applicability of the latter restriction to FRP bars requires further research and has been ignored in design. The former spacing requirements resulted in a maximum grid spacing of 88 mm for the columns tested in the current phase of research. One of the columns was designed to have a grid spacing of 88 mm and the other 175 mm. These spacing values and the total grid area of 192 mm^2 in each cross-sectional direction resulted in lateral drift capacity predictions of 1.9 % and 0.9 % based on Eq. 1 for spacings of 88 mm and 175 mm, respectively. The transverse reinforcement provided in columns was higher than that required by CSA S806-02 [2].

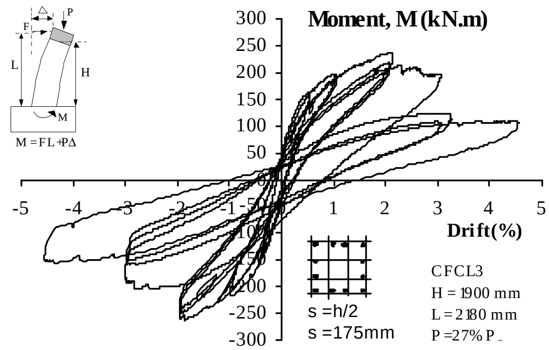
The beams were designed to have higher moment capacity in negative bending than positive bending, as is the case at the ends of beams designed for earthquake effects. The positive moment capacity was in excess of 50 % of the negative moment capacity, as required by seismic provisions of ACI 318-02 Code [3] and CSA A23.3-94 Standard [4] for steel reinforced concrete beams. The negative reinforcement consisted of 6 Pultral bars placed with an approximate spacing of 40 mm. This spacing allowed the placement of reinforcement in a single layer with sufficiently wide spacing to allow easy concrete placement. This amount of reinforcement resulted in a moment capacity that would be developed when both concrete crushing and FRP rupturing were approached simultaneously. Indeed, the condition of attaining 0.0035 compressive strain in extreme concrete fiber and experimentally observed rupturing strain of 0.013 in FRP would correspond to 0.39 % tension reinforcement ratio, which was exactly equal to the amount of negative reinforcement placed in the beam. Further increase in the number of bars could give sufficiently high reinforcement ratio to prevent FRP rupturing at ultimate load resistance, at the expense of congesting the beam with closely spaced bars. The beams had higher shear strengths, based on CSA S806-02 [2], than those corresponding to their flexural capacities. The transverse reinforcement was provided either at d/4 or d/2 spacing, with corresponding values of 90 mm and 180 mm, respectively, where “d” represents the effective depth of section, measured from the maximum compressive fiber to the centroid of tension reinforcement.

OBSERVED BEHAVIOUR AND TEST RESULTS

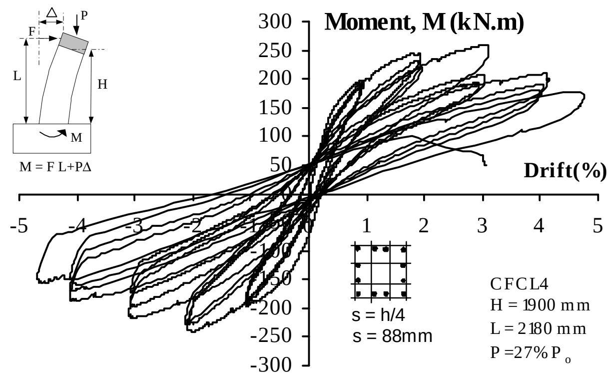
Hysteretic behaviour of columns and beams, in the form of moment-drift relationships, are shown in Figure 8. The figure indicates that Columns CFCL3 and CFCL4 both developed 2% drift capacities with little strength decay. CFCL4, with closer spacing of grids and amount of confinement reinforcement approximately equal to 62% of that required by CSA S806-02 [2], showed increased deformability, developing 3% lateral drift after 20% strength decay in the direction of original loading and 25% decay in the opposite direction. The column was able to sustain 4% lateral drift after 30% and 37% strength decay in two directions. The column failed during second cycle at 5% lateral drift when compression bars buckled and failed. The maximum strains recorded in longitudinal bars were 0.93% and 0.55% in tension and compression, respectively. The grids developed 0.5% strain in tension, a value higher than the conservative limit of 0.4% assumed in the CSA S806-02 [2] design expression.

Column CFCL3, with a wider grid spacing of 175 mm and about 31% of the confinement reinforcement required by CSA S806-02 [2] showed brittle behaviour shortly after 2% lateral drift, experiencing about a 50% drop in moment resistance at the end of 3% drift cycles. The maximum strains recorded in longitudinal bars were 0.76% and 0.50% in tension and compression, respectively. The confinement mechanism could not be fully activated in this column because of wide spacing of grids, and the recorded tensile strain in the grids was limited to 0.31%.

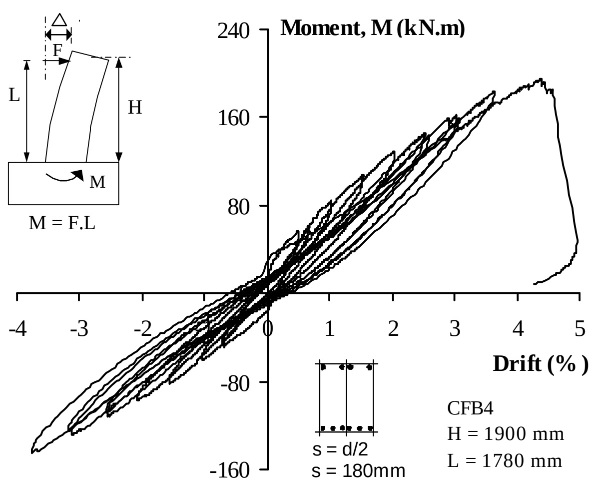
The beam hysteretic relationships are also shown in Figure 8. Both Beams CFB4 and CFB5 were companion specimens with approximately 1.8 m shear span, except for the grid spacing. CFB4 had 180 mm grid spacing, corresponding to d/2; and CFB5 had 90 mm grid spacing, corresponding to d/4. Both beams were designed to experience tensile rupturing of FRP bars at or shortly after the onset of concrete crushing. The hysteretic relationships indicate essentially elastic response, with gradual degradation of effective elastic stiffness due to progressive cracking under reversed cyclic loading. While both beams experienced 3% drift in the strong direction, the drift capacity was limited to 3% and 2% for CFB4 and CFB5, respectively, in the weak direction. The beams experienced softer response, as compared to the columns discussed earlier, because of the absence of accompanying axial compression. CFB4 experienced significant diagonal cracking on the side faces, in addition to progressively increasing flexural cracks.



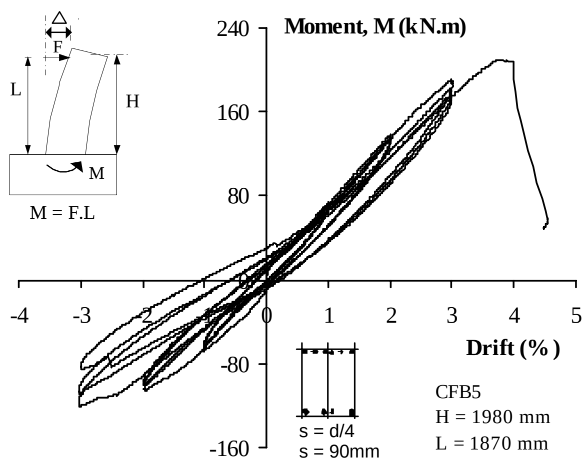
(a) Column CFCL3



(b) Column CFCL4



(c) Beam CFB4



(d) Beam CFB5

Figure 8 Experimentally recorded moment-lateral drift hysteretic relationships

However, diagonal cracking was controlled more effectively in CFB5 due to the reduced spacing of transverse grid reinforcement. This also resulted in controlled softening of CFB5 in the strong direction, with 185 kN.m of moment resistance at 3% drift, as compared to 156 kN.m moment resistance in CFB4 at the same levels of lateral drift. The strain readings indicated 0.0125 and 0.0120 tensile strains in longitudinal reinforcement of CFB4 and CFB5, respectively, before rupturing in tension at about 4% drift. The maximum compressive strain readings in FRP longitudinal bars were limited to 0.0038 and 0.0028 in CFB4 and CFB5, respectively. The transverse grid reinforcement developed maximum tensile strains of 0.0044 to 0.0040 in the beams.

CONCLUSIONS

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

- FRP reinforced columns and beams can be design to satisfy strength and deformability requirements of earthquake resistant structures. Tests performed under reversed cyclic loading indicate that column and beam drift capacities can be in excess of 3%.
- FRP reinforced concrete columns can be confined to develop inelastic deformations. The CSA S806-02 [2] requirements for confinement show good correlations with test data.
- FRP re-bars are capable of resisting significant compression and tension-compression cycles without any distress. The strength and elastic modulus of FRP bars in compression are approximately equal to 20% of the values in tension. The failure in tension and compression was both observed to occur at about 1.2% strain.
- The response of FRP reinforced concrete beams may be limited to elastic behaviour, with failure triggered by rupturing of tension reinforcement unless over-reinforced and confined by closely spaced transverse reinforcement. However, beams can develop significant deformability due to the softening caused by concrete cracking and low modulus FRP reinforcement, without having to develop significant inelasticity in compression concrete.
- Seismic design strategies for FRP reinforced concrete elements may be to design them remain elastic, with sufficient lateral deformability. The design approach may be improved by providing sufficient confinement for compression members by means of closely spaced transverse FRP reinforcement.
- Hysteretic behaviour of FRP reinforced concrete elements can be substantially different than that for steel reinforced concrete members. Inelastic response of FRP reinforced concrete elements can only occur in well confined and over-reinforced elements. Unloading branches of hysteresis loops aim at the origin, unlike those for steel reinforced concrete elements.

REFERENCES

1. Saatcioglu, M. and Razvi, S. R. “Displacement based design of reinforced concrete columns for confinement, ACI Structural Journal, Vol. 90, No.1. January 2002, pp.3-11.
2. CSA S806-02. “Design and construction of building components with fibre-reinforced polymers,” Canadian Standards Association, Rexdale Ontario, May 2002.
3. ACI Committee 318, “Building Code Requirement for Reinforced Concrete (ACI318-02) and Commentary (318R-02),” American Concrete Institute, Farmington Hills, Mich., 2002, 443pp.
4. CAN3-A23.2-94. “Design of Concrete Structures for Buildings,” Canadian Standards Association, Rexdale, Ontario, 1994, 199 pp.