Seismic Behavior of Beam-Column Joints Reinforced with GFRP Bars and Stirrups

Mohamed Mady¹; Amr El-Ragaby²; and Ehab El-Salakawy³

Abstract: Reinforced concrete beam-column joints are commonly used in structures such as parking garages and road overpasses, which might be exposed to extreme weathering conditions and the application of deicing salts. The use of the noncorrodible fiber-reinforced polymer (FRP) reinforcing bars in such structures is beneficial to overcome the steel-corrosion problems. However, FRP materials exhibit linear-elastic stress-strain characteristics up to failure, which raises concerns on their performance in beam-column joints in which energy dissipation, through plastic behavior, is required. The objective of this research project is to assess the seismic behavior of concrete beam-column joints reinforced with glass (G) FRP bars and stirrups. Five full-scale exterior T-shaped beam-column joint prototypes were constructed and tested under simulated seismic load conditions. The longitudinal and transversal reinforcement types and ratios are the main investigated parameters in this study. The experimental results showed that the GFRP-reinforced joints can successfully sustain a 4.0% drift ratio without any significant residual deformation. This indicates the feasibility of using GFRP bars and stirrups as reinforcement in the beam-column joints subjected to seismic-type loading. It was also concluded that, increasing the beam reinforcement ratio, while satisfying the strong column-weak beam concept, can enhance the ability of the joint to dissipate seismic energy. DOI: 10.1061/(ASCE)CC.1943-5614.0000220. © 2011 American Society of Civil Engineers.

CE Database subject headings: Beam columns; Joints; Seismic effects; Fiber reinforced polymer; Reinforcement.

Author keywords: Beam-column joints; Seismic loading; GFRP reinforcement.

Introduction

The fiber-reinforced polymer (FRP) reinforcement is currently being used as a viable alternative to steel in new concrete structures especially those in harsh environments. The main driving force behind this effort is the superior performance of FRP in corrosive environments attributable to its noncorrodable nature. However, the FRP materials exhibit linear-elastic stress-strain characteristics up to failure with relatively low modulus of elasticity [40–60 GPa for glass (G) FRP compared to 200 GPa for steel]. Moreover, they have different bond characteristics and relatively low strength under compressive and shear stresses. These mechanical characteristics raise concerns among researchers on the validity of using GFRP in structural members that require the inelastic behavior (ductility) of reinforcement. One example of these structural elements is beam-column joints in seismic regions [American Concrete Institute (ACI)-ASCE 352 2002]. The design philosophy of such joints is on the basis of providing the structure with an adequate ductile mechanism to dissipate the seismic energy. Such a ductile mechanism should be on the basis of a weak beam-strong column concept, while preventing shear damage in the joint zone. It is thought that the large deformation, exhibited by FRP material, may be beneficial in replacing yielding of steel and consequently, allow the FRP-reinforced concrete to adequately dissipate the seismic energy.

Very little research has been conducted to study the behavior of concrete columns and frame structures reinforced with FRP reinforcement, particularly subjected to seismic loading (Fukuyama et al. 1995; Sharbatdar et al. 2007; Hasaballa 2009). It was concluded that by using aramid, carbon, and glass FRP bars as flexural and shear reinforcement in concrete frames is feasible. However, owing to lack of data, none of the current FRP design codes and guidelines [Canadian Standards Association (CSA) 2002, 2009; ACI 2006] provides any recommendations on the seismic design of beam-column connections when FRP bars are used as primary reinforcement in both longitudinal and transversal directions.

This paper presents the experimental results of full-scale beam-column joints reinforced with GFRP bars and stirrups subjected to simulated seismic loads. The test parameters included the effect of beam reinforcement ratio and longitudinal and transversal reinforcement type.

Experimental Program

Test Specimens

A total of five full-scale beam-column joint prototypes were constructed and tested in the McQuade Structural Laboratory in the Department of Civil Engineering at the University of Manitoba. The first test specimen, SS, was reinforced with conventional steel bars and stirrups and used as a control specimen. The second specimen, GS, was reinforced with GFRP bars and steel stirrups. The remaining three specimens, GG-1, GG-2, and GG-3, were totally reinforced with GFRP bars and stirrups. Each prototype is simulating a beam-column connection of an exterior bay in a multibay,
multistory reinforced concrete moment-resisting plane frame. The span of the considered frame (bay length) is 4,700 mm with a story height of 3,650 mm. Each specimen represents an exterior connection between assumed points of contraflexural at midheight of columns and midspan of beams. Fig. 1(a) shows the overall dimension of a typical test prototype in which the beam is 2,350-mm-long with 350 × 450 mm in cross section and the column is 3,650-mm-high with cross section of 350 × 500 mm.

The beam, column, and joint of each specimen were designed separately according to the available design codes, manuals, guidelines and data in literature. Two Canadian codes, “Design of concrete structures,” CSA A23.3-04 (CSA 2004) and “Design and construction of building components with fiber-reinforced polymers,” CSA S806-02 (CSA 2002), were used for the design of these structural elements. In addition, the results of previous research for design of eccentric columns reinforced with FRP bars were also considered (Sharbatdar 2003; Choo et al. 2006; Sharbatdar et al. 2007).

For the steel-reinforced specimen, SS, the reinforcement ratio, bar size, and spacing of longitudinal and transversal reinforcement were calculated on the basis of CSA A23.3-04 (CSA 2004) code provisions. Table 1 gives the reinforcement details of specimen SS.

For GFRP-reinforced specimens, according to CSA S806-02 (CSA 2002), clause 8.2.1, the beam flexural failure should be initiated by crushing of concrete in the compression zone before the rupture of FRP bars in tension. In other words, beam section should be over-reinforced ($\rho_{FRP}/\rho_b > 1.0$). The selected $\rho_{FRP}/\rho_b$ ratios were 1.2, 1.7, and 2.9 for specimens GG-1, GG-2, and GG-3, respectively.

For the shear design, because no seismic provisions are available in FRP codes (CSA 2002), the concepts provided by the steel design code CSA A23.3-04 (CSA 2004) were applied. Clause 21.3.4.2 (CSA 2004) specifies that under seismic loading and to accommodate the concrete deterioration attributable to cracking, the contribution of concrete to shear strength is ignored where the plastic hinge could develop. In other words, the beam transverse reinforcement is required to carry the whole shear applied to the beams.
Following this concept, the shear reinforcement area, $A_v$, was determined according to clauses 8.4.4.4 to 8.4.4.6 of the CSA (2002). Also, the spacing of shear reinforcement in the beam, $s_v$, was determined according to confinement requirement (CSA 2004). Clause 21.3.3.2 of CSA (2004) specifies that $s_v$ shall not exceed the smallest of: quarter of the section depth, eight times the diameter of the smallest longitudinal bar, 24 times the diameter of the hoop bars or 300 mm.

Similarly, no provisions for the design of FRP-reinforced concrete columns subjected to eccentric loading (axial load and bending moment) were available. Therefore, the design of the columns was on the basis of the recommendations of previous research (Choo et al. 2006; Sharbatdar 2003; Sharbatdar et al. 2007). The column axial load capacity, $P_{ro}$, is predicted using Eq. (1)

$$P_{ro} = 0.85\phi f_y(A_p - A_{frp}) + \phi\varepsilon_{frp} E_{frp} A_{frp}$$

in which the value of the strain in FRP under concentric compression, $\varepsilon_{frp}$, is limited to 0.002. Also, the elastic modulus of FRP bars in compression, $E_{frp}$, is assumed to be equal to 25% of their modulus in tension, $E_{frp}$. In addition, the interaction diagram of the column reinforced with FRP bars is obtained by applying the procedures provided by Choo et al. (2006). The design of the GFRP-reinforced beam-column specimens also satisfied the strong column-weak beam concept with the same flexural strength ratio as the steel-reinforced one, equals 2.0.

The provisions of CSA S806-02 (CSA 2002), clause 12.7.1 to calculate the required amount of GFRP hoops, $A_{FH}$, in the transverse direction to confine a column were used [Eq. (2)]. Also, clause 21.4.4.3 (CSA 2004) was followed to obtain the adequate stirrup spacing in the column, $s_c$, determined as the smallest of: one quarter of the minimum member dimension or six times the diameter of the smallest longitudinal bar

$$A_{FH} = 14sh_0 \frac{f_y}{f_{FH}} \left( \frac{A_p}{A_p + A_{frp}} - 1 \right) \frac{\delta}{K_c} \frac{P_f}{P_{ro}}$$

in which $[(A_p/A_c) - 1] \geq 0.3$; $(P_f/P_{ro}) \geq 0.2$; and $K_c = 0.15 \sqrt{(h_s/s)} (h_s/s)$. Furthermore, the shear capacity of the joint $V_p$ was obtained according to clause 21.5.4.1 (CSA 2004), as follows:

$$V_p = 1.3\lambda \phi \varepsilon_f \sqrt{f_y A_f}$$

The obtained shear reinforcement was maintained along the whole length of the column. Table 1 and Fig. 2 show the reinforcement details of the tested specimens.

**Materials Properties**

All test specimens were constructed by using normal weight, ready-mixed concrete with maximum aggregate size of 20 mm. All test prototypes were cast and wet-cured in the laboratory for 7 days. For

![Fig. 2. Construction details of beam-column joint test prototypes (photos by M. Mady)](https://example.com/fig2)

**Table 2. Mechanical Properties of Reinforcing Bars Provided by Manufacturer**

<table>
<thead>
<tr>
<th>Bar type</th>
<th>Bar diameter (mm)</th>
<th>Bar area (mm²)</th>
<th>Modulus of elasticity (GPa)</th>
<th>Tensile strength (MPa)</th>
<th>Ultimate strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel #20M</td>
<td>19.5</td>
<td>300</td>
<td>200</td>
<td>$f_y = 400$ (460)$^a$</td>
<td>$\varepsilon_y = 0.2$ (0.23)$^a$</td>
</tr>
<tr>
<td>Steel #15M</td>
<td>15.9</td>
<td>200</td>
<td>200</td>
<td>$f_y = 400$ (460)$^b$</td>
<td>$\varepsilon_y = 0.2$ (0.23)$^b$</td>
</tr>
<tr>
<td>Steel #10M</td>
<td>11.3</td>
<td>100</td>
<td>200</td>
<td>$f_y = 400$</td>
<td>$\varepsilon_y = 0.2$</td>
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<tr>
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<td>728</td>
<td>1.53</td>
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<tr>
<td>GFRP #16</td>
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<td>198</td>
<td>48</td>
<td>751</td>
<td>1.56</td>
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<tr>
<td>GFRP #13</td>
<td>12.7</td>
<td>126</td>
<td>46</td>
<td>590$^a$</td>
<td>1.28</td>
</tr>
<tr>
<td>GFRP #10</td>
<td>9.5</td>
<td>71</td>
<td>45</td>
<td>642$^b$</td>
<td>1.42</td>
</tr>
</tbody>
</table>

$^a$ $f_y$ and $\varepsilon_f$ are the yield stress and yield strain of the used steel given by manufacturer, respectively. The numbers between parentheses are the experimentally obtained values.

$^b$ Strength of straight portion of the bent GFRP bar.
ease of construction, all specimens were cast in a horizontal layout (on the side). The targeted 28-day concrete compressive strength was 30 MPa, however, the actual concrete properties were determined on the basis of standard cylinder tests. On the day of specimen testing, the obtained average concrete compressive strength was approximately 32 MPa for the five test prototypes.

Two types of reinforcing bars were used in this study: CSA grade 400 deformed steel bars and sand-coated GFRP bars (Pultrall Inc. 2007). The mechanical properties of these reinforcing bars as provided by the manufacturers (used in design) are listed in Table 2. However, standard tests were performed on steel and GFRP samples to confirm the manufacturers' values. The obtained yield strength and yield strain of the used steel were 460 MPa and 0.0023, respectively. For GFRP reinforcement, the obtained properties were very close to those provided by the manufacturer.

**Test Setup and Instrumentations**

All specimens were tested while the column was lying horizontally and the beam was standing vertically, 90-degree rotated from the actual position. The cyclic load was applied at the tip of the beam by using a fully dynamic actuator, as shown in Fig. 3. Also, a 1000-kN-capacity hydraulic jack was positioned horizontally to apply a concentric load on the column. The two ends of the column were restrained against both vertical and horizontal displacements, whereas their rotations were allowed (hinged boundary conditions).

For each specimen, a total of 20 electrical resistance strain gauges were attached to the reinforcing bars and stirrups at critical locations to measure strains [Figs. 1(b) and 1(c)]. Also, eight linear variable displacement transducers (LVDTs) were used to measure the beam and column rotations, and the joint distortion (Fig. 4). In addition, two high accuracy (±0.001 mm) LVDTs were installed.
The first was at the position of first crack to measure crack width, whereas the second was at the end of one of the beam bars to measure slippage, if any. Furthermore, two load cells were used to monitor the column axial load and the vertical reaction at one of the column ends.

**Test Procedure**

For all specimens, an axial load of 800 kN (15% of the column capacity) was applied to the column and maintained constant throughout the test. The loading procedure for all specimens consisted of two phases. It started with a load-controlled phase followed by a displacement-controlled one. During the load-controlled phase, two load cycles were applied; the first cycle was to determine the cracking load, whereas the second cycle was corresponding to the service limit state. The cracking load was calculated to be approximately 25 kN and therefore, one cycle with a peak load of 30 kN was applied to all specimens. The peak load of the second cycle was different for each specimen, on the basis of code definitions. The service level for the steel-reinforced specimen (SS) was considered at 60% of steel yielding strain (CSA 2004). However, for the GFRP-reinforced specimens, the service level was considered at 25% of the ultimate tensile strength of GFRP bars (CSA 2009). The specimens were loaded until reaching the calculated service strain levels, as shown in Fig. 5(a). Table 3 gives the calculated and observed loads at different loading stages. The steel-reinforced specimen, SS, was tested first to obtain the beam yielding load, $P_y$, and its corresponding displacement, $\Delta y$. Once the cracking and service loads were obtained, the values of $P_y$ and $\Delta y$ were calculated considering linear behavior of load-deflection graph up to yielding. The obtained value for $\Delta y$ was 16.5 mm.

The second phase followed the recommendations of the American Concrete Institute (ACI) Committee 374 Report on the acceptance criteria for moment frames based on structural testing (ACI 2005). This criteria has been adopted by many researchers (Hakuto et al. 2000; Ghobarah and El-Amoury 2005; Chun et al. 2007). In this second phase, the seismic loading was applied in several steps under displacement-control mode at a quasi-static rate of 0.01 Hz. Each loading step consisted of three identical displacement cycles applied to the beam end, following the history shown in Fig. 5(b). In the first step, the displacement amplitude was equal to $\Delta y$. Then, the

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cracking load (kN)</th>
<th>Service load (kN)</th>
<th>Ultimate capacity (kN)</th>
<th>Mode of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Theoretical</td>
<td>Experimental</td>
<td>Theoretical</td>
<td>Experimental</td>
</tr>
<tr>
<td>SS</td>
<td>25</td>
<td>25</td>
<td>64</td>
<td>60</td>
</tr>
<tr>
<td>GS</td>
<td>21</td>
<td>20</td>
<td>49</td>
<td>58</td>
</tr>
<tr>
<td>GG-1</td>
<td>21</td>
<td>25</td>
<td>35</td>
<td>35</td>
</tr>
<tr>
<td>GG-2</td>
<td>21</td>
<td>22</td>
<td>71</td>
<td>89</td>
</tr>
<tr>
<td>GG-3</td>
<td>21</td>
<td>18</td>
<td>49</td>
<td>57</td>
</tr>
</tbody>
</table>

**Fig. 6.** Mode of failure (photos by M. Mady): (a) specimen SS; (b) specimen GS; (c) specimen GG-1; (d) specimen GG-2; (e) Specimen GG-3
displacement amplitude of each subsequent step was a multiplier of $\Delta y$; 1.3($\Delta y$), 1.8($\Delta y$), 2.5($\Delta y$), 3.35($\Delta y$), 4.17($\Delta y$), 5.37($\Delta y$), and 6.7($\Delta y$). The corresponding drift ratios for these displacements, calculated as the horizontal displacements of the beam end divided by the distance between the point of load application and the column centerline, are shown in Fig. 5(b). After the completion of the fourth seismic loading step (drift ratio of 1.85%), one load-controlled cycle with a peak load equal to the service load was applied to evaluate the loss in stiffness attributable to load cycles, if any. Identical loading scheme, considering $\Delta y$, was applied to GFRP-reinforced specimens.

**Test Results and Observations**

**General Behavior and Modes of Failure**

For the control steel specimen, SS, cracks developed in the plastic hinge zone (extended from the column face to a distance equal to...

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**Fig. 7.** Hysteretic load-drift ratio relationship: (a) specimen SS; (b) specimen GS; (c) specimen GG-1; (d) specimen GG-2; (e) specimen GG-3

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J. Compos. Constr., 2011, 15(6): 875-886
the beam depth), which in turn resulted in causing a permanent damage in this area. No significant cracks appeared in the column or in the joint area. The observed mode of failure for that specimen was a concrete crushing at 5.0% drift ratio attributable to excessive deformation in the beam (buckling of steel bars in compression), as shown in Fig. 6(a). For specimen GS, the failure occurred gradually, starting by spalling of concrete cover at 4.0% drift ratio, followed by complete concrete crushing combined with partial rupture for one GFRP bar at 5.0% drift ratio. The first bar was completely ruptured (test stopped) after completing the first cycle of the 6.5% drift ratio. No significant cracks appeared in the column or in the joint area, as shown in Fig. 6(b).

The observed mode of failure for specimen GG-1 ($\rho_{FRP} = 1.2\rho_b$) was a concrete crushing, immediately followed by rupture of the beam GFRP bars at 5.0% drift ratio, as shown in Fig. 6(c). Compared to specimen GG-1, the failure of specimen GG-2 ($\rho_{FRP} = 1.7\rho_b$) was quite gradual, starting with a concrete crushing at 4.0% drift ratio, followed by a rupture of the beam bars at 5.0% drift level [Fig. 6(d)]. Also, no significant cracks appeared in the column or in the joint in both specimens, GG-1 and GG-2. Factors such as slab flange, transverse beams, and higher-than-design mechanical properties of concrete will result in higher concrete strains at crushing, which may lead to rupture of GFRP bars before concrete crushing (brittle tension failure). Therefore, a well over-reinforcement ratio is recommended. Fortunately, to satisfy the severability requirements, typical flexural reinforcement ratios of 1.5-2.5 times the balanced ratio are commonly used in GFRP-reinforced section. This range of flexural reinforcement may be adequate to avoid the brittle failure. In contrary, for specimen GG-3, a large number of diagonal shear cracks started to appear in the joint area at 3.1% drift level. At 4.0% drift ratio, the cracks became wider and started to propagate toward the far edge of the column, accompanied with concrete cover spalling from the beam. Finally, specimen GG-3 failed in the joint area at 5.0% drift ratio (test stopped), as shown in Fig. 6(e).

Compared with specimen SS, more uniform cracks were observed along the full beam length for all GFRP-reinforced specimens as shown in Fig. 6. Also, it was observed that between loading steps (no load), these cracks would almost close without significant residual deformations. This was expected because of the elastic behavior and the bond characteristics of the used sand-coated GFRP bars. Furthermore, on the basis of the results of a previous study by the writers (Mady et al. 2010), it was found that an embedment length of 20 times bar diameter was not enough to prevent the GFRP beam bars from slippage under reversal loading. Therefore, the actual embedment length used in this study was 24 times bar diameter for specimens GG-2, GG-3, and GS, and 30 times bar diameter for specimen GG-1. According to the LVDT readings, no slippage of the beam bars was observed before failure for all GFRP-reinforced specimens. This indicates that embedment length of 24 times bar diameter ($24d_b$) seems to be adequate to

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**Fig. 8.** Stiffness-drift ratio relationship

**Fig. 9.** Maximum strain in beam longitudinal bars-drift ratio relationship

**Fig. 10.** Maximum strain in column longitudinal bars-drift ratio relationship

**Fig. 11.** Maximum strain in beam stirrups-drift ratio relationship
transfer the beam bars forces to the joint under cyclic loading. Further investigations are required regarding the bond behavior of the GFRP bars subjected to cyclic loading.

**Hysteretic Behavior**

Fig. 7 shows the load on the beam end at each seismic loading step versus lateral drift. Except for the last loading step, the first two cycles in each loading step were eliminated for clarity. For the steel-reinforced specimen, SS, the load-drift relationship indicates that the steel-reinforced specimen reached the maximum lateral load carrying capacity (178 kN) at the first cycle on drift ratio of 5.0%, and then this capacity started to decrease. The experimentally obtained capacity is higher than the design capacity (137 kN) because of the strain hardening phenomenon and higher yield strength of steel reinforcement than that given by the manufacturer (used in design). Considering the experimentally obtained yielding stress for the used steel bars, the corresponding ultimate capacity would be 156 kN. In addition, up to 1.0% drift level, neither significant pinching length nor stiffness loss was observed. However; starting at 1.35% drift level up to the maximum reached drift level of 5.0%, pinching length was increasing gradually, which indicates

![Stirrups strain profile across the joint: (a) specimen SS; (b) specimen GS; (c) specimen GG-1; (d) specimen GG-2; (e) specimen GG-3](image)

**Fig. 12.** Stirrups strain profile across the joint: (a) specimen SS; (b) specimen GS; (c) specimen GG-1; (d) specimen GG-2; (e) specimen GG-3
the capability of the specimen to dissipate energy, in addition to a gradual decrease in the overall stiffness, as shown in Fig. 7(a).

For specimen GS, the load-drift relationship indicates that the specimen reached the maximum lateral load carrying capacity (150 kN) at the first cycle of the 5.0% drift ratio, and then this capacity started to decrease. This obtained capacity was higher than the design capacity by approximately 9%. In addition, no significant pinching length appeared until failure started, as shown in Fig. 7(b). For specimen GG-1, the beam failed at a drift level of 5.0%. However, the lateral load capacity continued to increase up to failure at 131 kN. This obtained capacity was higher than the design capacity by approximately 8%. In addition, no significant pinching length appeared through the whole test and the behavior of the specimen was linear-elastic without any significant stiffness degradation observed up to failure, as shown in Fig. 7(c).

For specimen GG-2, the load-drift relationship indicates that the joint started to fail at a drift level of 4.0%, in which the lateral load capacity started to decrease. The maximum measured capacity was 150 kN, which is higher than the design capacity by approximately 9%. The behavior of the specimen remained linear-elastic without significant pinching length up to the 3.1% drift level. Differently from specimen GG-1, a gradual decrease in the overall stiffness combined with an increase in pinching length was observed up to failure [Fig. 7(d)]. For specimen GG-3, the load-drift relationship indicates that the specimen reached the maximum lateral load carrying capacity (173 kN) at the first cycle on drift ratio of 4.0%, and then this capacity started to decrease. This obtained capacity was higher than the design capacity by approximately 12%. In addition, no significant pinching length appeared up to drift level 3.1%, as shown in Fig. 7(e).

The observed loss in stiffness, on the basis of Fig. 7, was also confirmed by the service cycles that were performed after each seismic step. Fig. 8 shows the stiffness-drift ratio relationship, in which the stiffness of the specimen is calculated as the slope of the line connecting the two peaks of load-lateral deflection relationship. At 4.0% drift level, around 65% losses in the tangent stiffness was observed (compared to the initial stiffness) for the steel-reinforced specimen, SS. However, for the GFRP-reinforced specimen, GG-1, only 22% stiffness losses was recorded up to failure. For specimens GS, GG-2, and GG-3, the recorded losses in stiffness (before failure) were approximately 55, 62, and 48%, respectively. For the GFRP-reinforced specimens, the behavior remained basically linear-elastic until failure initiated.

In addition, it appears that the low modulus of elasticity for the GFRP reinforcement led to reducing the overall stiffness of the specimen, which is considered an advantage on the overall structural behavior. It is true that the lower stiffness for the GFRP-reinforced frame will result in higher displacements, however, in the mean time, it will result in higher natural period, which is inversely proportioned to the design spectral acceleration. This means that the total base shear of the structure with lower stiffness will be less. Further investigation is required to evaluate the overall seismic behavior of FRP-reinforced concrete frames. The ACI acceptance criteria (ACI 2005) requires that the joint should be able to retain its structural integrity and at least three-quarter of its ultimate capacity through peak displacements equal to or exceed a story drift ratios of ±3.5%, whereas the corresponding drift ratio required by the National Building Code of Canada (NBCC 2005) for earthquake resistant columns is only 2.5%. All tested specimens were successfully able to sustain drift ratios higher than the values required by both ACI and NBCC.

**Strain Measurements**

Fig. 9 shows the measured strains in the beam longitudinal reinforcing bars at the column face for the tested specimens. For the steel-reinforced specimen, SS, the measured strains in the beam bars remained elastic up to a 1.0% drift ratio. Then, the maximum measured strains (13,400 microstrains, approximately 5.8 times the yielding strain) in the beam bars were reached at 1.85% drift level. Afterward, they decreased to approximately 5,300 microstrains and stayed constant until the end of the test. This can be attributable to the yield penetration into the joint because the measured strains on beam bars inside the joint exceeded the yield strain at 1.85% drift level. Accordingly, the local slippage from the joint reduced the strain values for the beam bars at the column face (plastic hinge zone). For all the GFRP-reinforced specimens, the GFRP beam bars remained, as expected, linear-elastic up to failure, with maximum measured strains of approximately 15,250, 14,400, 13,350, and 9,950 microstrains for specimens GS, GG-1, GG-2, and GG-3, respectively. These strain values show that GFRP bars

![Fig. 13. Cumulative energy dissipation-drift ratio relationship](image)

![Fig. 14. Main components to total beam drift angle](image)
exhibited deformations in the same order as steel at failure of specimens, which indicates the validity of replacing yielding of steel with large-elastic deformations of GFRP reinforcement. For all test specimens except GS, all strain gauges on longitudinal reinforcing beam bars malfunctioned after the 4.0% drift loading step. For the GS specimen, the strain gauges were properly functioning until the 5.0% drift ratio, in which the maximum measured strain was 17,700 microstrains.

Fig. 10 shows the measured strains in the longitudinal column reinforcement of the tested specimens. For all specimens, the strains in the column bars remained elastic up to failure. For specimen GG-3, the maximum strain developed in the column bars was approximately 5,900 microstrains (3.5 times the steel-reinforced specimen), but still much less than the rupture strain of the FRP material (15,300 microstrains). This indicates the applicability of the column design approach followed in this study.

For the beam transverse reinforcement, the maximum strains developed in the beam stirrups at failure were approximately 1,900, 1,500, 3,650, 3,600, and 4,200 microstrains for specimens SS, GS, GG-1, GG-2, and GG-3, respectively, as shown in Fig. 11. However, at 2.50% (NBCC limit) the maximum observed strain was 2,750 microstrains for specimen GG-3, which is less than 4,000 microstrains, the CSA-S806-02 limit. Furthermore, the maximum measured strain in the transverse reinforcement inside the joint for the specimens SS, GS, GG-1, GG-2, and GG-3 were 2,400, 1,950, 4,250, 3,900, and 9,400 microstrains, respectively, as shown in the stirrups strain profile across the joint (Fig. 12). Both types of shear reinforcement (GFRP and steel) provided adequate confinement for the beams and columns to exceed the permitted drift ratio by CSA standards.

For specimen GG-3, the developed strains in the joint stirrups were 9,300 microstrains, much higher than the allowable limit of 4,000 microstrains (CSA-S806-02). As a result, wider cracks were developed in addition to disassembling the C-branch stirrups from one another. In other words, no more confinement was provided for the joint, which in turn led to the joint failure of this specimen. From section analysis, as the outer layer of beam reinforcement reached a strain level of 9,950 microstrains, the developed strains...
in the inner layer of the beam reinforcement were calculated to be 8,850 microstrains. Such strains induce a shear stress of approximately 5.3 MPa on the joint, which is in good agreement with the joint capacity of 5.6 MPa predicted by both ACI-318-08 (ACI 2008) and CSA-A23.3-04 (CSA 2004).

**Cumulative Energy Dissipation**

Fig. 13 shows the cumulative energy dissipated by test specimens during reversed cyclic loading and drift ratio. The cumulative energy dissipation, or cumulative energy absorbed, was calculated by summing up the dissipated energy in successive load-displacement cycles. Throughout the test, it was obvious that the absorbed energy of the steel-reinforced specimen is much higher than the GFRP-reinforced specimens. For example, at 2.50% drift level, the cumulative absorbed energy for specimen SS was 23 kN m, whereas for the GG-2 specimen, it was only 7.5 kN m (approximately 1/3). However, it was observed that increasing the beam reinforcement ratio, which resulted in changing the mode of failure of the beam section to concrete crushing (specimen GG-2) instead of the balanced failure mode (GG-1), led to enhance the ability of the joint to dissipate the seismic energy through utilizing the inelastic behavior of concrete. Increasing the beam reinforcement ratio would increase the shear forces transmitted to the joint and may result in joint failure (specimen GG-3). Therefore, the beam reinforcement ratio should be limited to the available shear capacity of the joint. Although this lower energy dissipation for the GFRP-reinforced specimens is considered a disadvantage, the joint will regain its original shape after removing the loads, thus requiring minimum amount of repair. On the other hand, specimen GG-2 with GFRP stirrups showed 20% more cumulative energy dissipation compared to specimen GS with steel stirrups. This indicates the validity of using GFRP stirrups in such connections subjected to reversal cyclic loading.

**Rotation Measurements**

The measured drift values can be divided into four main components, as shown in Fig. 14. These components are (1) rotation in the anticipated beam plastic hinge zone (for steel-reinforced joint), (2) rotation attributable to local slippage and large strains developed in the beam bars within the joint, (3) rotation attributable to overall column rotation, and (4) joint distortion. Each pair of LVDTs was installed to measure the rotation owing to one of the main drift components (Figs. 3 and 4). Then, the contribution of each rotation component can be determined in relation to the total beam drift angle (rotation). In FRP-reinforced joints, the plastic hinge zone is presented by the large-elastic deformation exhibited by the GFRP bars, which could be called a "virtual plastic hinge."

As shown in Fig. 15, rotation attributable to beam bar strains in the virtual plastic hinge zone contributed most to the total drift angle for all tested joints. Up to 2.50% drift ratio (NBCC limit), the contribution of the virtual plastic hinge rotation was approximately 35–40% of the total drift angle. However, rotation attributable to beam bar slippage and large strains in the joint was approximately 20–30% of the total drift angle, whereas the column rotation combined with the joint distortion effect was not more than 12% of the total drift angle. These obtained ratios are in agreement with the observed damage on the tested specimens up to the code limit (no significant damage was observed in the column or the in joint area). All remaining rotations from the total beam drift in Fig. 15 are probably attributable to unmeasured factors, i.e., beam cracks outside the anticipated plastic hinge zone.

**Conclusions**

On the basis of the experimental results, the following conclusions can be drawn:

1. GFRP bars and stirrups can be used as reinforcement in the beam-column joints subjected to seismic loading conditions. The GFRP bars were capable of resisting reversal tension-compression cycles with no problems.

2. The GFRP-reinforced joints can be designed to satisfy both strength and deformability requirements. The tested GFRP-reinforced concrete beam-column joints reached 4.0% drift capacity safely with insignificant damage. The obtained drift capacities are more than the 2.5% required by the NBCC (NBCC 2005) and the 3.5% required by the ACI (ACI 374.1 2005).

3. For the steel-reinforced joint, the plastic hinge was developed in the beam at the face of the column. However, for the GFRP-reinforced joints and because of large-elastic deformation, a virtual plastic hinge was developed away from the column face, which was spread over a longer length of the beam. In other words, the local yielding of steel reinforcement was partially substituted with the large-elastic deformations exhibited by GFRP bars.

4. The low modulus of elasticity for the GFRP reinforcement led to reducing the stiffness of the tested specimens, which resulted in attracting lower forces from the acting drifts. Considering the overall behavior of an FRP-reinforced frame, even though the lower stiffness will require higher displacement demand, it will reduce the expected base shear forces.

5. For GFRP-reinforced specimens, no slippage of the beam bars was observed before failure. In other words, an embedment length of 24 times bar diameter (24db) seems to be adequate to transfer the forces in the beam bars to the joint under cyclic loading. Further investigations are required regarding the bond behavior of the GFRP bars subjected to cyclic reversal loading.

6. Following the 4.0% drift cycle, the measured residual strains in the GFRP longitudinal beam reinforcement were negligible. However, at the same drift level, the steel-reinforced joint exhibited much larger residual strains. This indicates that, surviving an earthquake event, GFRP-reinforced joints would remain functional with a minimum required amount of repair, if any.

7. According to the used design concepts, the GFRP stirrups were adequate to provide the required confinement for both beams and columns. Specimen GG-2 with GFRP stirrups obtained cumulative dissipation energy higher than that obtained from the identical specimen, GS, with steel stirrups by approximately 20%.

8. For the GFRP-reinforced joints, as long as the joint is safe under the applied shear stresses, increasing the beam reinforcement ratio can enhance the ability of the joint to dissipate the seismic energy through utilizing the inelastic behavior of concrete.

**Acknowledgments**

The writers wish to express their gratitude and sincere appreciation for the financial support received from the Natural Science and Engineering Research Council of Canada (NSERC), through the Discovery and Canada Research Chair programs. Also, the equipment provided by a Canada Foundation for Innovation (CFI) grant is greatly appreciated. The help received from the technical staff of the McGuade Heavy Structural Laboratory in the Department of Civil Engineering at the University of Manitoba is also acknowledged.
Notation

The following symbols are used in this paper:

- $A_c$ = core area of column measured to the centerline of the hoops;
- $A_{FH} = \text{minimum transverse reinforcement for confinement purposes;}$
- $A_{frp} = \text{Aramid fiber reinforced polymer;}$
- $A_j = \text{joint cross-sectional area, taken equal to column cross-sectional area;}$
- $E_{frp} = \text{compressive modulus of elasticity of FRP longitudinal reinforcement;}$
- $f_c' = \text{concrete compressive strength after 28 days;}$
- $f_{FH} = 0.004E_{frp};$
- $h_c = \text{column core dimension of direction under consideration;}$
- $P_f = \text{actual applied axial load;}$
- $P_{ta} = \text{maximum column axial load resistance;}$
- $s_b = \text{spacing of transverse stirrups in beam;}$
- $s_c = \text{spacing of transverse stirrups in column;}$
- $s_1 = \text{spacing of tie legs in cross-sectional plane of column;}$
- $V_{fr} = \text{maximum shear resistance of joint;}$
- $\bar{\delta} = \text{design lateral drift ratio, shall not be less than 3\%;}$
- $\lambda = \text{concrete density factor, equal to one for normal weight concrete;}$
- $\rho_b = \text{FRP bars balanced reinforcement ratio;}$
- $\phi_h = \text{FRP bars reinforcement ratio;}$
- $\phi_c = \text{capacity reduction factor for concrete adopted by CSA standards;}$
- $\phi_F = \text{capacity reduction factor for FRP reinforcement adopted by CSA standards.}$

References


ACI Committee 440. (2006). “Guide for the design and construction of concrete reinforced with FRP bars.” *ACI 440.1R-06*, American Concrete Institute, Farmington Hills, Detroit, 44.


