Seismic Performance of Beam-Column Joints Reinforced With GFRP Headed Bars

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ABSTRACT

Concrete beams, slabs and recently columns reinforced with Fibre-Reinforced Polymers (FRP) reinforcements have shown considerable deformability under monotonic and fatigue loading. The behaviour of FRP bars under tension-compression load reversals in reinforced concrete (RC) beam-column joints and frame structures has not been fully investigated yet. Furthermore, concerns still remain regarding the ability of FRP-RC frame structures to dissipate energy in seismic loading events due to the elastic-linear behaviour of the FRP reinforcement. This paper attempts to partially fulfill this gap by investigating the structural performance and ultimate capacity of concrete beam-column connections reinforced with glass FRP bars. Two full-scale exterior beam-column joint (T-shaped) prototypes were constructed and tested under simulated seismic load conditions. The main test parameter is the detailing of the beam longitudinal bars within the joint by using either straight-headed bars or bent bars. Test results are presented in terms of load-drift ratio and load-strains in longitudinal reinforcement relationships. The experimental results showed a superior performance for the straight-headed bars over the bent ones.

KEYWORD

Seismic performance, beam-column joints, GFRP headed bars, GFRP bent bars, drift ratio
1. INTRODUCTION

The corrosion of steel reinforcement is a major factor in limiting the service life of concrete structures especially those in harsh environments such as bridges and parking garages which represent a significant percentage of the infrastructure located in North America. In recent years, the use of non-corrodible glass fibre-reinforced polymer (GFRP) reinforcement in such structures has emerged as a cost effective and reliable alternative to steel. GFRP reinforcement, in general, offers many advantages over the conventional steel such as high strength-to-weight ratio, favourable fatigue performance and high electro-magnetic transparency. However, the GFRP reinforcement exhibits linear-elastic behaviour up to failure with relatively low modulus of elasticity (40 - 60 GPa compared to 200 GPa for steel). Moreover, they have different bond characteristics, relatively low strength under compression stresses, and some concerns still remain about their performance under load reversal conditions [1]. In seismic zones, moment-resisting frames require sufficient ductility to dissipate the seismic energy. Due to the non-ductile linear elastic characteristics of FRP reinforcements, concerns still exist among researchers on the validity of using FRP in such structural members that require the inelastic behaviour (ductility) of reinforcement. The research work done on the behaviour of concrete beam-column joints, reinforced with FRP bars, is still in its early stages with very few studies [2, 3, 4]. These previous studies revealed that using FRP bars as flexural and shear reinforcement is feasible. However, current design codes and guidelines for FRP-reinforced concrete structures [1,5,6] have little, if any, seismic provisions due to lack of data and research in this area.

The focus of this research is to investigate the effect of the beam reinforcement detailing within the joint on the ultimate capacity and performance of the beam-column joints in RC frames subjected to seismic loading. The most critical zone in moment-resistant frames is the exterior joints due to the unsymmetrical conditions and restraints associated with providing limited anchorage length for the beam longitudinal reinforcement. As a common practice in case of exterior beam-column joints reinforced with conventional steel, top and bottom beam longitudinal reinforcement are usually bent inside the joint to enhance their anchorage and prevent slippage. Moreover, using headed bars is also a viable alternative of 90-degree hooked bars in joints. In case of FRP reinforcement, some restrictions still apply when using bent bars. The current manufacturing process of FRP bent bars limits the size and length of the bent bar as well as the tensile strength, which is reduced by up to 50% at the bend location [5] compared to pultruded straight bars. On the other hand, the performance of GFRP headed-bars in beam-column joints has not been investigated yet.

2. THE EXPERIMENTAL PROGRAM

2.1 Test Specimens

This study is part of an extensive experimental program currently in-progress in the McQuade Heavy Structural Laboratory at the University of Manitoba to investigate the performance of GFRP-RC frames subjected to seismic loading. The experimental program includes construction and testing of a number of full-size exterior (T-shape) beam-column joint prototypes. In this paper, two full-size beam-column joint prototypes (I-B and I-H) were reinforced with GFRP bars and steel stirrups. Both specimens had the same concrete dimensions, column reinforcement, and beam transverse reinforcement as shown in Fig. 1(a). The variable in the two specimens is the beam flexural reinforcement using headed or bent bars in specimen I-H and I-B, respectively, as shown in Figure 1(b, c).

Since the current design codes and guidelines for FRP-reinforced concrete structures provide little to none seismic provisions, one of the main challenges of this research was to develop a design procedure for concrete beam-column connections reinforced with GFRP bars. Both specimens were designed to follow the strong column-weak beam concept with a column-to-beam flexural strength ratio larger than 1.0. The deformable concrete crushing failure in the beam section near the joint was considered to achieve this concept by providing a longitudinal reinforcement ratio, in the beam, larger than the balanced one. In specimen I-B, the beam was reinforced with 4-No.20 GFRP bent bars (top & bottom). Due to current manufacturing limitations, the available total length of a GFRP bent bar is approximately 2200 mm including the bent portion. Therefore, the longitudinal beam bars were spliced using a lap splice length of 40 times the bar diameter (800 mm).
Considering the strength reduction factor of 0.75 for GFRP bent bars [5,7,8] the calculated flexural capacity of the beam and column in specimen I-B was 268.0 and 202.0 kN.m, respectively, considering the axial load applied on the column. Accordingly, the resulted column-to-beam flexural strength ratio was 1.51.

For specimen I-H, no strength reduction factor was applied to the GFRP tensile strength of the straight headed bars, only 4-No.15 GFRP at top and bottom were used as beam longitudinal reinforcement. The calculated flexural capacity of the beam and column in specimen I-H was 231.0 and 183.0 kN.m, respectively, considering the axial load applied on the column. Accordingly, the resulted column-to-beam flexural strength ratio was 1.58. Table 1 summarizes the design characteristics of each specimen, and the calculated stresses and strength based on the concrete strength on the day of testing.

Table 1 Design characteristics of specimens (I-H) and (I-B)

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>I-H</th>
<th>I-B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam End Anchorage</td>
<td>Headed Bars</td>
<td>90-degree hooks</td>
</tr>
<tr>
<td>Calculated Bar stress (MPa)</td>
<td>796</td>
<td>619</td>
</tr>
<tr>
<td>$\rho_{FRP} / \rho_{bal}$ *</td>
<td>1.78</td>
<td>1.68</td>
</tr>
<tr>
<td>Flexural capacity (kN.m)</td>
<td>231</td>
<td>268</td>
</tr>
<tr>
<td>Overall Flexural strength ratio</td>
<td>1.58</td>
<td>1.51</td>
</tr>
<tr>
<td>Joint shear stress (MPa)</td>
<td>4.08</td>
<td>4.76</td>
</tr>
<tr>
<td>Concrete Strength (MPa)</td>
<td>30.5</td>
<td>34.0</td>
</tr>
<tr>
<td>Mode of failure</td>
<td>Beam compression failure</td>
<td></td>
</tr>
</tbody>
</table>

$\rho_{FRP} / \rho_{bal}$ is the ratio between the provided reinforcement ratio to the balanced reinforcement ratio.

2.2 Material Properties

Both test specimens were constructed using normal-weight, ready-mixed concrete with a targeted 28-day concrete compressive strength of 30 MPa and a maximum aggregate size of 20 mm. The obtained average concrete compressive strength on the day of testing was 30.5 and 34.0 MPa for specimens I-H and I-B, respectively.

The GFRP bars used in this study (Schöck-ComBAR™) had a deformed surface (ribs) and a high modulus of elasticity. Table 2
lists the mechanical properties of the GFRP reinforcing bars.

**Table 2 Properties of GFRP bars**

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Bar Diameter (mm)</th>
<th>Elastic Modulus (GPa)</th>
<th>Ultimate Strength (MPa)</th>
<th>Ultimate Strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No.15</td>
<td>15.9</td>
<td>60</td>
<td>1100</td>
<td>1.83</td>
</tr>
<tr>
<td>No.20</td>
<td>19.1</td>
<td>50</td>
<td>875</td>
<td>1.75</td>
</tr>
</tbody>
</table>

2.3 Test Set-up and Instrumentations
Specimens were tested while the column was lying horizontally and the beam was standing vertically; 90-degree rotated from the actual condition, as shown in Fig. 2. A fully dynamic actuator was utilized to apply the reversed quasi-static cyclic loading to the tip of the beam following the loading scheme [9,10] shown in Fig. 3. A hydraulic jack was used to apply a constant axial compression force to the columns during testing equal to approximately 15% of the column ultimate capacity (650 kN). For each specimen, a total of twenty electrical resistance strain gauges were attached to reinforcing bars and stirrups at critical locations to measure strains. Eight linear variable displacement transducers (LVDTs) were used to measure the beam and column rotations, as well as the joint distortion. Also, a high accuracy (± 0.001 mm) LVDT was installed to measure the slippage of longitudinal beam reinforcement in specimen I-H, if any. In addition, two load cells were used to monitor the column axial load and the vertical reaction at one of the column ends. A data acquisition system was used to monitor and record the instrumentations readings during the test.

3. TEST RESULTS AND OBSERVATIONS

3.1 Load-Lateral Drift Response
Plots of the hysteresis diagrams represent the relationship between the applied lateral load and the drift ratio of the beam tip. The drift ratio was calculated as the horizontal displacement of the beam end divided by the distance between the point of load application and the column centreline (i.e. 2200 mm). As shown in Fig. 4(a), the measured hysteresis loops for specimen I-H demonstrated stable response up to a drift ratio of 4.0% then severe load degradation was observed approximately after a drift ratio of 4.5%, during the 5.0% drift ratio loading step. It also shows that the beam sustained a flexural capacity of 256 kN.m just prior to concrete crushing. This implies that the specimen achieved a flexural capacity of 10% more than the calculated capacity in a stable manner. Specimen I-H had an elastic performance with no decrease in stiffness up to a drift ratio level of 4.0%. Test results showed that specimen I-H reached the same level of loading in both loading directions. However, for specimen I-B, as shown in Fig. 4(b), the measured hysteresis loops demonstrated stable response up to a drift ratio of 1.5%. Figure 4(b) also shows that after a drift ratio of 2.0%, the specimen showed no increase in flexural resistance in the positive loading direction (+ve values of lateral loads are when the actuator pushes the beam tip) contrary with the opposite loading direction which showed increasing flexural resistance up to 240 kN.m. Moreover, specimen I-B exhibited considerable degradation in flexural resistance within the same loading step after 2.0% drift ratio. The loss of flexural resistance was 40% and 10% in the +ve and –ve loading directions, respectively. This is attributed to the loss of bond between concrete and the bars as a result of bar ribs shearing off, which in turn diminished the mechanical bond between the concrete and the bars.
3.2 Ultimate Capacity and Mode of Failure

In both specimens, the formation and propagation of cracks were marked after each loading step. Damage was generally expressed by the initiation and propagation of flexural and shear cracks. Further damage and failure by concrete crushing was also observed. Figure 5 shows the cracking patterns at different loading steps for specimen I-H. In general, specimen I-H had a stable behaviour up to 4.0% drift ratio with minimal cracking in the joint area. In addition, specimen I-H showed no diagonal shear cracks before 2.0% drift ratio loading level. Up to 1.5% drift ratio, only flexural cracks were observed in the beam near the column face, while no shear cracks were observed in the joint. Diagonal shear cracks started to show up in the joint at 2.0% drift ratio. The penetration depth and the number of cracks slightly increased with the increasing drift ratio till failure as shown in Figs. 5(a) through 5(c). Specimen I-H reached the design flexural capacity of the beam (231 kN.m) at drift ratio level of 4.0%. This capacity was maintained till 4.5% drift ratio, which was observed while applying the 5.0% drift ratio loading step as shown in Fig. 4. Test results showed 1.0 mm slippage in beam longitudinal bars after completing the last cycle in 4.0% drift ratio loading level. In the first cycle of this loading step, just at 4.5% drift ratio (i.e. 128 kN of lateral loading), the beam exhibited sudden drop in the flexural resistance due to loss of bond strength. However, the specimen was able to resist 50% of its flexural capacity even after the slippage took place.

Regarding specimen I-B, Fig. 6 shows the cracking patterns at different loading steps. Specimen I-B had a stable behaviour up to 1.5% drift ratio where flexural cracks were observed in the beam near the column face with no cracking in the joint area. At 2.0% drift ratio level, the specimen exhibited a difference in flexural resistance between the positive and negative loading direction where the specimen showed no increase in flexural resistance to the positive loading direction. This difference gradually increased with loading till it reached 66.0% of the resistance capacity recorded against negative loading direction at the end of the test. One diagonal shear crack showed up in the joint at 2.0% drift ratio and remained stable till the end of the test. At 3.0% drift ratio, vertical splitting cracks were observed in the concrete cover of the beam bars, which in turn forced the concrete cover to fail as shown in Figs. 6(a) through 6(c). This can be attributed to loss of bond as explained before in Section 3.1 above. Also, it is worth mentioning that major wide cracks were observed during loading starting at 3.0% drift ratio. These cracks were getting wider with the increase in drift till failure. The maximum flexural resistance achieved by specimen I-B was 90% of the design flexural capacity of the beam (268 kN.m) at drift ratio level of 5.0% and was recorded in the –ve loading direction as shown before in Fig. 4.
3.3 Lateral Drift-Strain Relationship

Figure 7 shows the maximum strains developed in beam longitudinal reinforcement at the column face for both specimens. In both specimens, the strain increased linearly with the increase of the drift ratio due to the linear-elastic behaviour of the GFRP reinforcement. For specimen I-B, unfortunately, following the 2.0% drift ratio, no more strains could be recorded due to malfunctioning of strain gauges at that location. At drift ratio level of 2.0%, specimen I-B had no increase in flexural resistance due to concrete-to-bar bond loss, which in turn reduced the strains in that side; however, the opposite side exhibited increase in strains up to 10,800 micro-strains. Specimen I-H also exhibited linear increase in strains up to 12,700 micro-strains at a drift ratio of 4.0%. The difference in strains between the two specimens in the same drift ratio is attributed to the difference in modulus of elasticity of the bars (i.e. 50.0 GPa for bent bars and 60.0 GPa for headed bars).

Figure 8 shows the strain distribution along the beam longitudinal reinforcement in the vicinity of the joint area for specimen I-H in which headed bars were used to provide anchorage for beam longitudinal bars. The contribution of the heads to develop the strength of the bar was evaluated by mounting a strain gauge on the bar just before the head (i.e. 300 mm away from the column face inside the joint). The ratio between the developed strain just before the head to the one on the column face reflects the percentage of force carried by bearing head. Accordingly, as shown in Fig. 8, the head contribution started at 3.0% drift ratio where the detected strain at the head (i.e. 3,400 micro-strain) was 34.0% of the one recorded at the column face (i.e. 10000 micro-strain). This value continued to increase with the loss of bond inside the joint. For instance, at 4.0% drift ratio,
the bearing resistance of the head reached 65.0% of the tensile force developed at the face of the column. At this level, the maximum strain in the bar was 12,500 micro-strain. Therefore, the tensile force in the bar was 750 MPa (i.e. \( \varepsilon \times E = 12,500 \times 10^{-6} \times 6,000 = 750 \) MPa). Accordingly, 488 MPa was bearing contribution of the head and 262 MPa was resisted by bond between the bar and the concrete. The maximum contribution of the head was reached in the first cycle of the 5.0% loading step; however, the contribution couldn’t be calculated due to malfunctioning of the strain gauge on the column face.

Fig. 7 Maximum strains–drift ratio relationship for beam bars

Fig. 8 Strain profile on beam longitudinal bars for specimen I-H

3.4 Stiffness-Drift Relationship
Figure 9 shows the stiffness-drift ratio relationship. In this figure, the stiffness of the specimen is calculated as the slope of the line connecting the two peaks of load-lateral displacement relationship. It can be noticed that at the service state loading limit (cracked section properties), the stiffness of specimen I-B was 15.0% higher than that of specimen I-H. This can be attributed to the higher concrete compressive strength of specimen I-B. Both specimens had the same stiffness approximately at 1.5% drift ratio. Then afterwards, specimen I-B exhibited dramatic loss of stiffness in the positive loading. Meanwhile, the opposite side of specimen I-B had stiffness close to the one of specimen I-H till 4.0% drift ratio. At the end of the test, the stiffness of specimen I-H had a dramatic decrease due to slippage of beam bars from the joint.

Fig. 9 Stiffness–drift ratio relationship

5. CONCLUSIONS

Based on the experimental results, the following conclusions can be drawn:

(1) Behaviour of specimen I-B was affected greatly by slippage of beam longitudinal bars which resulted in losing 40% of the designed flexural capacity of the beam in one direction of loading.

(2) Specimen I-H, with headed-bars achieved a flexural capacity of 10% higher than the design one. While, specimen I-B, with bent-bars, reached only 90.0% of the design capacity.

(3) Specimen I-H had a more stable cracking pattern than that of specimen I-B. Spalling of concrete cover started at 4% drift ratio for specimen I-B, meanwhile this occurred at 5% drift ratio for specimen I-H. Compared to specimen I-H, the area of spalling concrete cover was double for specimen I-B.

(4) In headed bars, the contribution of head-bearing to carry the tensile stresses in beam longitudinal bars reached 65.0% at drift ratio 4.0%.
ACKNOWLEDGEMENT

The authors 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 Canada Research Chairs program. Also, the equipment provided by a Canada Foundation for innovation (CFI) grant is greatly appreciated. The GFRP reinforcement generously provided by Schoeck Canada Inc. is greatly appreciated. The help received from the technical staff of the McQuade Heavy Structural Laboratory in the department of civil engineering at the University of Manitoba is also acknowledged.

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