INFLUENCE OF REINFORCEMENT DETAILING ON THE BEHAVIOR OF BEAM-COLUMN JOINTS REINFORCED WITH GFRP BARS

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Abstract
Implementing the non-corrodible glass fibre-reinforced polymer (GFRP) reinforcement in reinforced concrete (RC) infrastructure is a viable alternative to avoid steel-corrosion problems. However, the behaviour of GFRP bars in tension-compression reversed cycles in RC frame structures has not been well investigated yet. Furthermore, the elastic-linear behaviour of the GFRP reinforcement up to failure makes the ability to dissipate energy of frame structures in seismic loading events questionable. Therefore, this study attempts to partially fulfill this gap by investigating the structural performance and ultimate capacity of concrete beam-column connections reinforced with GFRP bars. In this paper, four full-scale exterior beam-column joint prototypes were constructed and tested under simulated seismic load conditions. Specimens were reinforced with GFRP bars. The test variable is the beam end anchorage (increased column depth, beam stubs and headed bars) of the beam longitudinal bars within the joint. Test results are presented in terms of load-drift ratio relationship, ultimate capacity and mode of failure. The study concluded that using beam stubs or headed bars are viable solution wherever geometric constraints do not allow for deep columns.

KEYWORDS
GFRP, beam-column joints, seismic loading, reinforcement detailing.

INTRODUCTION
The fibre reinforced polymer (FRP) reinforcement is currently being used as an innovative material in new concrete structures especially those in harsh environments such as bridges and parking garages. The main driving force behind this effort is the superior performance of FRP in corrosive environments due to its non-corrodible nature. FRP reinforcements, in general, offer many advantages over the conventional steel such as high strength-to-weight ratio, favourable fatigue performance and high electro-magnetic transparency. However, the FRP material exhibits linear-elastic behaviour up to failure with relatively low modulus of elasticity (40 - 65 GPa for glass FRP “GFRP” and 110 - 140 MPa for carbon FRP “CFRP” compared to 200 GPa for steel). Moreover, they have different bond characteristics, relatively low strength under compression stresses, and some concerns still exist about their performance under stress reversal conditions. 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 (Fukuyama et al. 1995; Said and Nehdi 2004; Sharbatdar et al. 2007; Hasaballa et al. 2011). These previous studies revealed that using FRP bars as flexural and shear reinforcement is feasible. However, the GFRP specimens showed lack of energy dissipation when compared to their steel counterparts. A viable solution to this issue may be the use of hybrid seismic resistant systems in which steel-reinforced concrete (RC) shear walls can provide lateral load resistance and absorb the seismic energy while using GFRP-RC moment resisting frames as the main structural system to support gravity loads. The presence of steel RC shear walls does not eliminate the need of the adjoining GFRP-RC frame members to share part of the seismic loads and lateral drifts through their seismically-induced deformations. Accordingly, GFRP-RC moment resisting frames should
properly designed and detailed. Moreover, current design codes and guidelines for FRP-reinforced concrete structures (CSA 2002; ACI 2006; CSA 2009) have little, if any, seismic provisions due to lack of data and research in this area. Also the anchorage behaviour of FRP straight bars under reversed load cycles is still questionable. The objective of this research is to investigate the effect of the beam longitudinal reinforcement end anchorage within the joint on the ultimate capacity.

EXPERIMENTAL PROGRAM

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 presented in this paper included the construction and testing of four full-size exterior (T-shape) beam-column joint prototypes as shown in Figure 1(a). The column depth was variable as shown in Figure 1(a) to generate different end anchorage conditions. In case of using straight FRP bars in exterior beam-column joints, the column depth may be governed by the demand for larger anchorage depth of beam longitudinal reinforcement rather than what is required to carry the applied loads. In this study, two different approaches were investigated to provide adequate anchorage for beam longitudinal reinforcement when straight GFRP bars are used. The first approach was to increase the embedment length of beam longitudinal bars either by extending the beam length beyond the column (beam stub) or by increasing the column depth. The second approach is to use headed-end bars as longitudinal beam reinforcement to reduce the required embedment length. To perform this investigation, one control specimen, S-35 - with a relatively small column depth of 350 mm was used. It is worth mentioning that the 350-mm column depth was selected to produce a column-to-beam flexural strength ratio just above 1.0. For the first approach, two specimens, BS-55 and S-50, were used. BS-55 included a 200-mm beam stub, while S-50 had a deeper column of 500 mm. This reinforcement configuration resulted in embedment lengths for specimens BS-55 and S-50 of 30 $d_b$ and 33 $d_b$, respectively, where $d_b$ is bar diameter. For the second approach, one specimen, H-40, utilizing headed-end bars and a slightly increased column depth (400 mm, to accommodate the bar head) was used. Details of headed bars are shown in Figure 1(b). Table 1 shows the column depth of each specimen.

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. All specimens were designed to follow the strong column-weak beam concept with a column-to-beam flexural strength ratio larger than 1.0 in which the failure is governed by the flexural capacity of the beam rather than of the column. 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. Table 1 shows the design characteristics of test prototypes. The beams in all test prototypes had a constant concrete dimensions and were designed to achieve the same flexural capacity. As shown in Table 1, the beam in specimens S-35, S-50 and BS-55 was reinforced with 5-No.15 (15.9 mm diameter) sand-coated GFRP longitudinal bars as top and bottom reinforcement, each. Regarding specimen H-40, it is worth mentioning that there were no sand-coated GFRP headed bars commercially available at the time this study started to be used in this specimen. Accordingly, ribbed GFRP headed bars were used. The beam in specimen H-40 was reinforced with 4-No.15 ribbed GFRP-headed bars as top and bottom reinforcement, each. To achieve the same level of flexural capacities of the first three specimens, the design of the beam in specimen H-40 resulted in using a lower number of bars than that of the other specimens as shown in Table 1. This was mainly attributed to the higher strength and tensile elastic modulus of the headed bars as shown in Table 2.
Figure 1. Configuration of test Prototypes

Table 1. Design characteristics of test specimens

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>S-35</th>
<th>S-50</th>
<th>BS-55</th>
<th>H-40</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>End anchorage</td>
<td>Straight Bars</td>
<td>Beam stub</td>
<td>Headed Bars</td>
<td></td>
</tr>
<tr>
<td>Long. reinforcement</td>
<td>5 No. 15</td>
<td>4 No. 15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reinforcement surface</td>
<td>Sand coated</td>
<td>Ribbed</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Calculated bar stress (MPa)</td>
<td>665</td>
<td>685</td>
<td>645</td>
<td>805</td>
</tr>
<tr>
<td>$\rho_{frp}/\rho_{bal}$ *</td>
<td>1.25</td>
<td>1.18</td>
<td>1.3</td>
<td>1.78</td>
</tr>
<tr>
<td>Flexural capacity (kN.m)</td>
<td>242</td>
<td>250</td>
<td>235</td>
<td>234</td>
</tr>
<tr>
<td>Overall</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flexural strength ratio</td>
<td>1.12</td>
<td>2.00</td>
<td>1.12</td>
<td>1.58</td>
</tr>
<tr>
<td>Concrete strength (MPa)</td>
<td>33</td>
<td>35</td>
<td>31</td>
<td>31</td>
</tr>
</tbody>
</table>

* $\rho_{frp}/\ rho_{bal}$ is the ratio between the provided reinforcement ratio to the balanced reinforcement ratio

Material Properties

Test specimens were constructed using normal-weight, ready-mixed concrete with a targeted concrete compressive strength of 30 MPa. The obtained average concrete compressive strength on the day of testing is shown in Table 1. Two types of GFRP bars were used; sand-coated bars (Pultrall 2009) and ribbed-deformed bars (Schoeck Canada 2011). Table 2 lists the mechanical properties as given by the manufacturers.

Table 2. Mechanical properties of GFRP bars

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Bar Diameter (mm)</th>
<th>Surface</th>
<th>Tensile Elastic Modulus (GPa)</th>
<th>Ultimate Tensile Strength (MPa)</th>
<th>Ultimate Tensile Strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No.15</td>
<td>15.9</td>
<td>Sand-coated</td>
<td>48</td>
<td>751</td>
<td>1.56</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Ribbed</td>
<td>64</td>
<td>1100</td>
<td>1.88</td>
</tr>
</tbody>
</table>

Test Set-up, Instrumentation and Procedure

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 Figure 2 (Hasaballa et al. 2011). A fully dynamic
actuator was utilized to apply the reversed quasi-static cyclic loading to the tip of the beam following the loading scheme shown in Figure 3 (ACI 2005; Ghobarah and El-Amoury 2005). 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. For each specimen, a total of twenty electrical resistance strain gauges were attached to reinforcing bars and stirrups at critical locations to measure strains.

TEST RESULTS AND OBSERVATIONS

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 (Fig. 4). The drift ratio was calculated as the ratio between the horizontal displacement of the beam end to the distance between the point of load application and the column centerline. All specimens exhibited the characteristic elastic-linear behavior of the GFRP reinforced elements with almost no significant stiffness degradation up to a drift ratio of 4.0% except for the control specimen, S-35, which exhibited the failure after completing the 3.0% drift ratio loading stage. Also, up to 4.0% drift ratio, each specimen reached the same level of loading in both loading directions. Specimen S-50 failed before completing the 5.0% drift ratio-loading stage, while specimen BS-55 was able to complete that loading stage. In addition, both specimens showed no significant pinching length through the whole test. Regarding specimen H-40, it demonstrated stable response up to a drift ratio of 4.0% then exhibited a 30.0% sudden drop in load resistance at a drift ratio of 4.5% during the 5.0% drift ratio loading step. The comparison between the four responses shows that they all reached the same level of flexural capacity except specimen S-35 which did not sustain the design flexural capacity. However, specimens S-50 and S-35 reached their capacity at 5.0% contrary to 4.0% for specimen H-40, which means that specimen H-40 had stiffness 25.0% higher than specimens S-50 and S-35. This can be attributed to the higher elastic modulus of reinforcement used in specimen H-40. Also test results showed that the behavior of specimen BS-55 was identical to the one of specimen S-50 although that specimen S-50 had 67.0% higher flexural strength ratio.
The propagation of cracks was marked after each loading step. In general, all specimens exhibited concrete crushing failure in beam section except the control specimen, S-35, which exhibited pre-mature failure due to slippage of beam longitudinal bars as shown in Figure 5(a). Test results showed that specimen S-35 achieved 74.0% of the design flexural capacity of the beam. On the other hand, specimens S-50 and BS-55 exceeded the design flexural capacity in the beam by approximately 4.0% while exhibiting no anchorage failure of beam longitudinal reinforcement. For specimens S-50 and BS-55, uniform cracks distribution was observed along the whole beam length till the end of the test. The intensity of cracks did not increase in the virtual plastic hinge zone (near the column face) till the end of the 4.00% drift level (Figures 5-b and c). Also, no significant cracks appeared in the joint till the end of the test. The observed mode of failure for these two specimens was a concrete crushing in the beam immediately followed by rupture of the beam GFRP bars at 5.00% drift ratio. This may be attributed to the low reinforcement ratio ($\rho_{frp} / \rho_{bal} = 1.3$). Specimen H-40 had a stable behavior up to 4.0% drift ratio with insignificant cracking in the joint area. In addition, no diagonal shear cracks were observed before 2.0% drift ratio loading level. Specimen H-40 exceeded the design flexural capacity in the beam by approximately 8.0% at drift ratio level of 4.0%. This capacity was maintained till 4.5% drift ratio, which was recorded while applying the 5.0% drift ratio loading step as shown previously in Fig. 4(d). In the first cycle of this loading step, just at 4.5% drift ratio, the beam exhibited sudden drop in the flexural resistance due to partial loss of bearing strength of the headed bars. This can be attributed to a partial damage in the bearing heads. However, the specimen was able to resist 50% of its flexural capacity even after the slippage took place. Moreover, in specimen H-40, the headed bars sustained a tensile stress in the amount of approximately 800 MPa at the 4.0% drift ratio loading stage. It is worth mentioning that specimen H-40 did not exhibit the sudden failure due to bar rupture that specimens S-50 and BS-55 had. This can be attributed to the higher $\rho_{frp} / \rho_{bal}$ that the specimen H-40 had compared to the other two specimens as shown previously in Table 1. This reflects the influence of high levels of $\rho_{frp} / \rho_{bal}$ to control the mode of failure when the reinforcement slippage is prevented.
CONCLUSIONS

Four GFRP-reinforced beam-column joints were tested. In light of the experimental results, the following conclusions can be drawn:

- The control specimen, S-35 with the smallest column depth, exhibited pre-mature failure due to slippage of beam longitudinal reinforcement. Contrary, the other specimens achieved the design flexural capacity of the beam without anchorage failure.
- Beam stubs can be used to provide anchorage of beam longitudinal reinforcement without increasing the column depth. The hysteresis response of specimen BS-55 with a beam stub was identical to the one of specimen S-50 with increased column depth although specimen S-50 had 67.0% higher flexural strength ratio.
- GFRP headed bars can be considered as a viable alternative to provide anchorage and reduce the required development length for straight bars wherever geometric constraints do not allow for beam stubs or deep columns.
- The ratio between the provided reinforcement ratio to the balanced reinforcement ratio \(\frac{\rho_{frp}}{\rho_{bal}}\) controls the mode of failure when reinforcement slippage is prevented.
- The findings of this study are preliminary, however they demonstrate the significant impact of anchorage detailing of GFRP beam longitudinal reinforcement on the seismic behavior of beam column joints. Further research is needed to study the influence other parameters affecting the anchorage detailing such as confinement of the joint, shear stress level in the joint, use of GFRP bent bars, level of axial load on column and concrete strength.

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