Effect of Reinforcement Detailing on the Behavior of GFRP-RC Beam-Column Joints

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ABSTRACT: The behavior of FRP bars under tension-compression load reversals in RC beam-column joints and frame structures has not yet been fully explored. This research project is attempting to partially fulfill this gap by investigating the structural performance and ultimate capacity of concrete beam-column connections totally reinforced with FRP bars. A total of three full-scale exterior beam-column joint (T-shaped) prototypes were constructed and tested under simulated seismic load conditions. The main test parameter was the detailing of the beam longitudinal bars within the joint by using either straight bars or bent bars. Test results are presented in terms of load vs. drift ratio, and strains in longitudinal reinforcement. The experimental results showed a superior performance for the GFRP reinforcement when bar slippage within is prevented through proper detailing of the beam bars within the joint.

1 INTRODUCTION

The fiber 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 electromagnetic transparency. However, the FRP material exhibits linear-elastic behavior up to failure with relatively low modulus of elasticity (40 - 50 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 load reversal conditions (CSA 2002). 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 behavior (ductility) of reinforcement.

2 BACKGROUND

Very little research has been conducted to study the behavior of concrete columns and frame structures internally reinforced with FRP bars subjected to seismic loading. Fukuyama et al. (1995) noticed the elastic behavior of concrete frames reinforced with longitudinal aramid FRP bars until failed by concrete crushing. Sharbatdar et al. (2007) reported that exterior beam-column joints reinforced with carbon FRP longitudinal bars and carbon FRP grids were able to achieved a lateral drift ratio exceeds 3.0%. This drift capacity exceeds the 2.5% drift ratio recommended by the National Building Code of Canada (NBCC 2005).

The focus of this research is to further investigate the effect of the beam reinforcement detailing within the joint on the ultimate and service 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 bond strength to concrete,
prevent slippage, and to provide more confinement to the joint. In case of FRP reinforcement, some restrictions still exist when using bent bars. The FRP bars have to be formed to the required shape during the manufacturing process, which currently limiting the straight part length of a bent bar. Moreover, due to relatively low lateral strength, the tensile strength of FRP bars is reduced by up to 50% at the bend location (ACI 2006).

3 EXPERIMENTAL PROGRAM

3.1 Test specimens

An experimental program is carried out in the McQuade Heavy Structural Laboratory at the University of Manitoba to investigate the performance of GFRP reinforcement in 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. Three full-size beam column joint prototypes were constructed and tested. All test prototypes were reinforced with No. 16 (15.9 mm diameter) GFRP longitudinal bars for both beams and columns. Figure 1 shows the overall dimensions of a typical test prototype. Table 1 shows the details of test specimens.

![Figure 1. Geometry of beam-column joint specimens](image)

The first specimen (G1) had a column depth of 350 mm with straight top and bottom beam bars within the joint. This reinforcement configuration resulted in embedment length for the beam top and bottom reinforcement within the joint of 20 times bar diameter (20 $d_b$). The second specimen (G2) has the same column depth of 350 mm but the beam top and bottom longitudinal reinforcement was bent into the joint. Due to manufacturing limitations, the bent GFRP bars could not be produced in lengths longer than 1500 mm. Therefore, the beam bars were spliced 460 mm away from the joint face with a 640 mm splice length (40 $d_b$). The third specimen (G3) is similar to (G1); however the column depth was increased to 500 mm; achieving an embedment length of 30 $d_b$.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column depth (mm)</td>
<td>350</td>
<td>350</td>
<td>500</td>
</tr>
<tr>
<td>Beam reinforcement</td>
<td>5 No. 16</td>
<td>6 No.16</td>
<td>5 No.16</td>
</tr>
<tr>
<td>Column reinforcement</td>
<td>8 No.16</td>
<td>8 No.16</td>
<td>8 No.16</td>
</tr>
<tr>
<td>Actual/Balanced reinforcement ratio (Beam)</td>
<td>1.15</td>
<td>0.83</td>
<td>1.15</td>
</tr>
<tr>
<td>Column/Beam flexural strength ratio</td>
<td>1.12</td>
<td>1.12</td>
<td>2.00</td>
</tr>
</tbody>
</table>

3.2 Materials properties

All test specimens were designed and constructed using normal weight, ready-mixed concrete with a targeted 28-day concrete compressive strength of 35 MPa. The reinforcing bars used in this study are sand-coated GFRP V-RODTM (Pultrall Inc. 2007). Table 2 shows the mechanical properties of the used GFRP bars.

<table>
<thead>
<tr>
<th>Bar Type</th>
<th>Tensile Modulus (GPa)</th>
<th>Tensile Strength (MPa)</th>
<th>Ultimate Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Straight</td>
<td>48.2</td>
<td>751</td>
<td>0.0156</td>
</tr>
<tr>
<td>Bent*</td>
<td>39.5</td>
<td>512</td>
<td>0.0130</td>
</tr>
</tbody>
</table>

* Properties of straight portion of GFRP bent bar

3.3 Test set-up and instrumentation

All 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. 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 & El-Amoury, 2005, Chun et al., 2007). A hydraulic jack was used to apply a constant axial compression force to the columns during testing equal to 15% of the column ultimate capacity (670 kN for specimens G1 and G2 and 800 kN for specimen G3).

Several electrical resistance strain gauges were installed on the GFRP reinforcement, linear variable displacement transducers (LVDTs) to measure the beam and column rotations, the joint distortion, and crack width. A data acquisition system, monitored
by a computer was used to monitor and record the readings during the test.

Figure 2. Test set-up

Figure 3. Seismic loading Scheme

4 RESULTS AND DISCUSSIONS

4.1 General behavior and modes of failure

Generally, test results showed that the behavior of all test specimens was satisfactory according to the requirements of the National Building Code of Canada (NBCC 2005) for earthquake resistant structures. The three test specimens exceeded the 2.50% drift ratio before failure. All specimens exhibited a deformable behavior with minimal damage in the joint area. The failure of the three specimens took place in the beam near the column face with no damage to the joint area. Specimen G1 failed due to insufficient anchorage length of beam reinforcement (20\(d_b\)) at 3.0% drift ratio before reaching the design strength. Specimen G2 exhibited failure due to slippage at the spliced portion of the beam reinforcement at 3.0% drift ratio after achieving the design capacity. This indicates the feasibility of using GFRP bent bars inside the joint to avoid the slippage problem within the joint. For specimen G3, the observed failure was a concrete crushing immediately followed by rupture of the beam GFRP bars at 5% drift ratio, after achieving the anticipated design capacity.

4.2 Strain measurements

Figure 4 shows the measured strains in the beam flexural reinforcing bars at the column face for the three test specimens. The strains in the GFRP bars remain, as expected, linear-elastic up to failure with maximum measured strains of approximately 12,000, 15,800 and 15,000 micro-strains for G1, G2 and G3, respectively. It can be seen that the GFRP bars were capable of exhibiting large elastic strains which indicates the feasibility of using GFRP bars in such joints based on replacing typical yielding of steel reinforcement with the large elastic deformations of FRP. The maximum strain recorded in specimen G3 (15,000 micro-strains) is less than the ultimate strain, which is in good agreement with design procedure as G3 was designed to fail by concrete crushing; over-reinforced. However, specimen G1 did not achieve the targeted strain due to early slippage failure (not enough anchorage).

Figure 4. Maximum strain in beam longitudinal bars-drift ratio relationship

Moreover, the measured strains in the longitudinal column reinforcement of the three tested specimens remain elastic up to failure. The maximum strains developed in the column bars were around 3800, 4000 and 4900 micro-strain for G1, G2 and G3, respectively, which still much less than the rupture strain of the FRP material (15,600 micro strains).

4.3 Load-lateral drift response

Figure 5 shows the typical relationship for the horizontal load applied at the beam tip versus lateral drift (load-drift behavior). For all specimens, the lateral load capacity continued to increase up to failure. No significant pinching appeared through the whole test. It can be noticed that all specimens had elastic-stable behavior without any significant stiffness degradation observed up to failure. Figure 6 shows comparison of the behavior of each specimen before failure. For G1 specimen, the failure occurred after
completing 3 cycles at 3.0% drift ratio at a load of 90 kN due to slippage of beam bars. For specimen G2, the failure occurred during the first loading cycle at 3.0% drift ratio at a load of 118 kN, which is close to the design capacity (125 kN). For specimen G3, the joint failed at a drift level of 5.0% at 135 kN. This obtained capacity is close to the design capacity of 125 kN.

5 CONCLUSIONS

Based on the results of the tested specimens, the following conclusions can be drawn:

- When enough anchorage length within the joint is provided, it is evident that the GFRP-reinforced joint can safely reach 5.0% drift capacity under reversed cyclic loading. This drift capacity is more than the 2.5% required by the National Building Code of Canada (NBCC 2005).
- Test results showed that anchorage length of 30 times bar diameter for the beam straight top and bottom longitudinal reinforcement (specimen G3) is enough to prevent slippage (bond failure).
- It is feasible to use GFRP bent bars to avoid bar slippage within the joint provided that bent bars are spliced with a lap splice greater than 40 times bar diameter

6 ACKNOWLEDGEMENT

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7 REFERENCES


