THE USE OF ULTRA-HIGH STRENGTH CONCRETE (UHSC) WITH LONGITUDINAL GFRP REINFORCEMENT UNDER FLEXURE

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This research investigates the behavior of ultra-high strength concrete (UHSC) beams with GFRP reinforcement under flexure. The design parameters included the type of longitudinal reinforcement (GFRP or steel), as well as the reinforcement ratio (0.26%, 0.41%, and 0.67%). The beam specimens had a width of 185 mm, a depth of 250 mm, and a total span length of 2200 mm. All the beams were developed with 140 MPa concrete and tested in a four-point bending setup. The test results revealed that increasing the GFRP ratio improved the flexural capacity of the tested beams. By increasing the ratio from 0.26% to 0.41%, the moment capacity increased by 47.5%, whereas increasing the ratio from 0.26% to 0.67% increased the capacity by 76.4%. Moreover, the steel-reinforced beam reported a smaller moment capacity and mid-span deflection values as compared to its GFRP counterpart. Overall, the ACI 440 code provided a good description of the behavior of FRP reinforced beams developed with UHSC.

Keywords: Fiber reinforced polymers, RC beams, Flexural strength, Deflection.

1 INTRODUCTION

Corrosion of conventional steel bars can lead to premature degradation of reinforced concrete (RC) structures. It can severely limit the service life of RC structures and affect their durability. Materials, like fiber-reinforced polymer (FRP) bars, have emerged as a substitute to steel bars, particularly in harsh and aggressive surroundings. FRP bars are becoming more popular due to their non-corrosive nature, high tensile strength, and lightweight. However, FRP bars have a linear stress-strain relationship and have a low modulus of elasticity. Hence, the design of FRP reinforced members is performed in terms of serviceability limit states as they experience large deflections and crack widths. In addition, FRP standards and guidelines indicate that FRP reinforced beams should be designed as over-reinforced to provide the beams with some degree of ductility before complete failure. There are many commercially available types of FRP bars, such as glass FRP (GFRP), basalt FRP (BFRP), and carbon FRP (CFRP).

The behavior of beams reinforced with FRP bars under flexure (Abed and Alhafiz 2019, Alkhrisha et al. 2020) and shear (Abed et al. 2021a) was examined in previous studies. Some researchers used high strength concrete (HSC) with the FRP reinforced beams and reported that the HSC helped in exploiting the high tensile strength of the FRP reinforcing bars. Abed et al. (2021b) reported a 16% increase in the ultimate moments of BFRP reinforced specimens when HSC was used in place of normal strength concrete (NSC). Increasing the FRP ratio yielded in smaller deflection values and better cracking behavior. Adam et al. (2015) found that increasing the compressive strength of concrete from 25 to 70 MPa improved the flexural capacity, cracking
response, and the load-deflection behavior of GFRP reinforced beams. Similar results were reported by El-Nemr et al. (2013), where the use of HSC enhanced the flexural behavior of GFRP reinforced specimens. In another study by Abdelkarim et al. (2019), GFRP reinforced beams constructed with HSC had higher ductility indices and moment capacities than NSC counterparts.

Very few studies investigated the effect of using UHSC in beams reinforced with FRP bars. Goldston et al. (2017) tested six GFRP reinforced beams developed with HSC and UHSC under flexural loading. The use of UHSC improved the flexural capacity of over-reinforced beams but had an insignificant effect on that of under-reinforced beams. Nonetheless, the over-reinforced beams displayed some ductility before failure, whereas under-reinforced beams failed suddenly with no warning. However, the increase in concrete compressive strength led to larger deflection values. Yoo et al. (2016) evaluated the flexural response of GFRP reinforced beams made with fiber-reinforced concrete that had a compressive strength of 197.3 MPa. After cracking, the specimens maintained a stiff load-deflection behavior owing to the excellent fiber-bridging capacity of this type of concrete at the crack surfaces.

In this study, the flexural behavior of UHSC beams reinforced with GFRP bars was examined in terms of cracking response, failure modes, ultimate moment, and mid-span deflection. The experimental program consisted of three beams with varying GFRP ratios. One steel reinforced beam was also fabricated and tested under flexural loads. The test results of the cracking and ultimate moments were compared with analytical predictions using the ACI 440 code (ACI Committee 440 2015).

2 EXPERIMENTAL PROGRAM

2.1 Materials

The concrete considered in this study had a nominal compressive strength of 140 MPa. The concrete strength was evaluated on the day of testing using three cubes and three cylinders. The cylinders had a diameter of 150 mm and a height of 300 mm, whereas the cubes were 150 mm X 150 mm X 150 mm in size. The cube compressive strength was found to be 144 MPa in average and the cylinders had an average strength of 110 MPa. Sand-coated GFRP bars were provided by Galen in Russia. The GFRP bars had diameters of 8, 10, and 12mm. Steel bars of 12 mm diameter were also used as tensile reinforcement. The GFRP bars had an average elastic modulus of 44.9 GPa and tensile strength of 973 MPa. The yield strength of the steel bars was 575 MPa, while the elastic modulus had a value of 200 GPa.

2.2 Test Specimen and Setup

Three GFRP-reinforced beams were prepared and tested in addition to one steel reinforced beam. Table 1 provides the details of the tested beams. All the beams had a 185 mm wide and 250 mm deep rectangular cross-section. The total span of the beams was 2200 mm, with a 1900 mm clear span and 150 mm overhang on both sides of the beam. The concrete cover was 30 mm from the bottom, 25 mm from the top, and 27.5 mm from the sides. Figure 1 shows the cross-sectional dimensions and reinforcement details of the specimens. All the beams were designed as tension controlled with either two GFRP bars or two steel bars in the tensile zone. Steel stirrups of 10 mm diameter were used as transverse reinforcement throughout the shear spans and spaced evenly at 100 mm centers. Two steel bars with 10 mm diameters were used in the compression region of the beam as hunger for the stirrups.

The test specimens were tested in a four-point loading setup, as depicted in Figure 1. All the beams were loaded using a universal testing machine of 2000 kN capacity. A hydraulic actuator
was used to apply a constant load to a spreader beam. The load was then passed to two points with a distance of 400 mm between them, which is the maximum bending region. Cracks initiation and propagation were visually observed for all the test specimens. Mid-span deflections of the UHSC specimens were evaluated using linear variable differential transducers (LVDTs) fixed at the beam soffit. The actuator and LVDTs were linked to a data acquisition system, which captured the readings throughout the test.

**Table 1. Test matrix details.**

<table>
<thead>
<tr>
<th>Beam</th>
<th>Bar type</th>
<th>Bar size (mm)</th>
<th>Reinforcement area (mm²)</th>
<th>ρf (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2T8G</td>
<td>GFRP</td>
<td>8</td>
<td>100.5</td>
<td>0.26</td>
</tr>
<tr>
<td>2T10G</td>
<td>GFRP</td>
<td>10</td>
<td>157.1</td>
<td>0.41</td>
</tr>
<tr>
<td>2T12G</td>
<td>GFRP</td>
<td>12</td>
<td>253.4</td>
<td>0.67</td>
</tr>
<tr>
<td>2T12S</td>
<td>Steel</td>
<td>12</td>
<td>226.2</td>
<td>0.60</td>
</tr>
</tbody>
</table>

![Figure 1. Beam dimensions and reinforcement details.](image)

**3 RESULTS AND DISCUSSIONS**

**3.1 Cracking Behavior**

The UHSC beams were observed visually throughout the tests and the load at which the first crack appeared was registered. Table 2 summarizes the experimental cracking and ultimate moments. It is evident that the amount of tensile reinforcement had an insignificant effect on the cracking moment of the tested specimens. Moreover, the ACI 440 code predicted well the cracking moments of the UHSC specimens. The beams had an average experimental to predicted ratio of 0.95±0.06. All the beams displayed a similar cracking response, where flexural cracks appeared in the pure flexure region of the beams when the tensile strength of the concrete was exceeded. New flexural cracks formed closer to the support as the applied load increased, whereas the already-formed cracks propagated towards the compression zone of the concrete. At failure, the GFRP bars ruptured, and the cracks in the maximum bending region widened, but the concrete at the top was intact (see Figure 2).
Table 2. Experimental and predicted cracking and ultimate moments.

<table>
<thead>
<tr>
<th>Beam Designation</th>
<th>Experimental Mn (kN.m)</th>
<th>δ (mm)</th>
<th>Mcr (kN.m)</th>
<th>Exp./Pred. ACI Mn</th>
<th>Mcr</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>2T8G</td>
<td>22.8</td>
<td>35.7</td>
<td>12.81</td>
<td>1.20</td>
<td>1.02</td>
<td>TC</td>
</tr>
<tr>
<td>2T10G</td>
<td>37.0</td>
<td>41.3</td>
<td>11.43</td>
<td>1.05</td>
<td>0.91</td>
<td>TC</td>
</tr>
<tr>
<td>2T12G</td>
<td>51.0</td>
<td>50.6</td>
<td>11.17</td>
<td>1.10</td>
<td>0.89</td>
<td>TC</td>
</tr>
<tr>
<td>2T12S</td>
<td>30.0</td>
<td>25.2</td>
<td>12.44</td>
<td>1.18</td>
<td>0.99</td>
<td>TC</td>
</tr>
</tbody>
</table>

3.2 Flexural Capacity and Failure Modes

In this study, the UHSC beams were designed as tension-controlled. Thus, all the FRP reinforced beams failed due to the rupture of the reinforcement, as shown in Figure 2a. The failure was abrupt and sudden, without the beam showing any warning. The steel-reinforced beam failed due to the yielding of the steel bars (see Figure 2b). It can be seen from Table 2 that increasing the reinforcement ratio improved the flexural capacities of the beams. By increasing the reinforcement ratio from 0.26% to 0.41% (from 2T8G to 2T10G), the ultimate capacity increased by 47.5%. Likewise, increasing the GFRP reinforcement ratio from 0.26% to 0.67% (from 2T8G to 2T12G) resulted in a 76.4% improvement in the ultimate moment of the UHSC specimens.

Moreover, the GFRP reinforced beam (2T12G) reported a higher moment capacity by 51.9% as compared to its equivalent steel reinforced beam (2T12S). It can be concluded that using UHSC with GFRP reinforcement is more beneficial than with steel reinforcing bars due to the high tensile strength of the FRP bars. Similar deductions were made by Abed et al. (2021b), who reported that the flexural capacity of FRP reinforced HSC beams was 130% times greater than their steel counterparts.

The ratio of experimental to predicted ultimate moments of the tested specimens had an average value of 1.13 with a standard deviation of 0.07. The experimental values followed a similar trend to the predicted values of the ACI 440 code. However, the equation somewhat underestimated the actual moment capacities of the under-reinforced beams.

3.3 Moment-Deflection Relationship

Figure 3 shows the bilinear moment-deflection relationship of FRP reinforced beams. All the beams had identical behavior before cracking but experienced a reduction in the stiffness once
cracking occurred. The curves reveal that the reinforcement ratio had a notable impact on the post-cracking stiffness of the specimens. At the same moment level, increasing the FRP ratio increased the bending stiffness, which in turn resulted in lower midspan deflection values. The beam with the highest GFRP ratio (2T12G) displayed the largest deflection at failure. The curves also show that the steel-reinforced beam (2T12S) has better ductility than the GFRP reinforced beam (2T12G). At the ultimate moment, the 2T12S beam reported a significantly smaller midspan deflection (25.2 mm) as compared to the 2T12G beam (50.6 mm).

![Figure 3. Moment-deflection curves.](image)

4 CONCLUSIONS

In this study, the behavior of three UHSC beams with GFRP reinforcement and one beam with steel bars has been investigated. The main variable in consideration was the reinforcement ratio. It was found that the reinforcement ratio had a minor influence on the cracking moment of the beams. All the GFRP reinforced beams failed by the rupturing of the bars, which led to the widening of the cracks in the maximum bending region. The failure was very sudden and the beams showed no signs of ductility. The results revealed that the reinforcement ratio had a more pronounced effect on the ultimate capacity of the UHSC beams. Increasing the GFRP ratio from 0.26% to 0.67% increased the flexural capacity up to 76.4%. In addition, the steel reinforced beam reported smaller moment capacity and midspan deflection values than its GFRP counterpart. In general, the ACI 440 equations provided a good description of the behavior of GFRP under-reinforced beams developed with UHSC. It predicted well the cracking moments of the beams but somewhat underestimated their flexural capacities.

Acknowledgments

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