SHEAR FAILURE OF UHPC-FRP HYBRID BEAMS

Mina Iskander¹, Raafat El-Hacha² and Nigel Shrive³

Department of Civil Engineering, Schulich School of Engineering, University of Calgary
2500 University Dr. NW, Calgary, Alberta, T2N 1N4, Canada, Web Page: http://www.ucalgary.ca
Email: ¹mina.iskander@ucalgary.ca, ²relhacha@ucalgary.ca, ³ngshrive@ucalgary.ca

Keywords: Pultruded GFRP, Shear, Hybrid, UHPC, Externally bonded

Abstract

The Paper includes an experimental study focusing on the shear performance of a hybrid section built from glass fibre reinforced polymer (GFRP) rectangular hollow section, an ultra-high performance concrete (UHPC) flange and a bottom sheet made of steel reinforced polymer (SRP) or carbon fibre reinforced polymer (CFRP). This section has distinct structural strength compared to sections made from conventional materials. Eight specimens were tested by applying a point load 280 mm from one support over a clear span of 1120 mm to induce shear failure. All specimens failed similarly, indicating consistent behaviour for shear failure regardless of the changing bottom plate and flange dimensions. Failure happened by crack propagation at the corners of the GFRP rectangular section followed by cracking of the UHPC flange. Elastic analysis of the shear stresses at the failure point showed that high shear stresses are not the cause of this mode of failure. The cause of failure is interpreted using a simple finite element model as bending failure at the corners. Per this reason, proper design should be done for the corners in the case of unidirectional pultruded sections.

1. Introduction

Developing new construction materials has created new opportunities for structural engineering, retaining many of the advantages of conventional construction materials while minimizing the disadvantages. Here we present a quick overview of an experimental investigation into the shear capacity of a recently developed hybrid section constructed from three more recently available ultra-high performance materials. The results of this experimental study are published in details in ref.[1]. The section consists of a glass fibre reinforced polymer (GFRP) hollow box section with an ultra-high performance concrete (UHPC) flange and a bottom plate made of carbon fibre reinforced polymer (CFRP), as shown in Figure 1. This section has superior properties, compared to conventional materials. The flexural behaviour of this section configuration had been studied by [2, 3], while no one targeted the shear behaviour. Eight specimens were tested with a clear span of 1120 mm and a point load applied at 280 mm from one support to induce shear failure. Two main parameters were investigated, namely, the effects of flange dimensions and the type of bottom plate. The failure of all specimens tested was observed to be similar, indicating consistent behaviour for shear failure regardless of the changing parameters. The flange dimensions were found to influence the shear capacity of these beams whereas the type of bottom plate appeared to have no influence.

The static behaviour of beams of pultruded rectangular box sections of Glass FRP (GFRP) and their failure modes have been studied [4, 5]. A common failure mode was progressive tearing, where the vertical and horizontal walls (webs of the box section) separated at the corners of the section. The
propagation of cracks along the corners of such beams using an energy criterion was simulated numerically [6].

![Figure 1. UHPC-FRP hybrid section.](image)

2. Experimental programme and results

All specimens were composed of a pultruded GFRP hollow box section separating a cast-in-place flange of UHPC on its top face and a laminate of superior tensile strength material (either CFRP or SRP) on the bottom face. The external dimensions of the GFRP sections were 152.4 mm x 228.6 mm with an average wall thickness of 11.11 mm. Details of flange dimensions and bottom plate type and dimensions are provided in Table 1.

![Table 1. Test matrix and results.](image)

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Concrete flange Width (mm)</th>
<th>Thick. (mm)</th>
<th>Span (mm)</th>
<th>Bottom sheet</th>
<th>Width of sheet (mm)</th>
<th>Thickness of sheet (mm)</th>
<th>Maximum Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W15-1</td>
<td>153</td>
<td>52</td>
<td></td>
<td>CFRP</td>
<td>129</td>
<td>0.76</td>
<td>135.5</td>
</tr>
<tr>
<td>W15-2</td>
<td>154</td>
<td>50</td>
<td></td>
<td>SRP</td>
<td>127</td>
<td>1.23</td>
<td>178.0</td>
</tr>
<tr>
<td>W19-1</td>
<td>187</td>
<td>55</td>
<td>120</td>
<td>CFRP</td>
<td>76</td>
<td>1.0</td>
<td>259.1</td>
</tr>
<tr>
<td>W19-2</td>
<td>188</td>
<td>57</td>
<td>120</td>
<td>CFRP</td>
<td>75</td>
<td>1.0</td>
<td>256.0</td>
</tr>
<tr>
<td>W19-3</td>
<td>188</td>
<td>56</td>
<td></td>
<td>CFRP</td>
<td>76</td>
<td>1.0</td>
<td>316.6</td>
</tr>
<tr>
<td>W25-1</td>
<td>254</td>
<td>50</td>
<td>120</td>
<td>SRP</td>
<td>75</td>
<td>1.23</td>
<td>269.1</td>
</tr>
<tr>
<td>W25-2</td>
<td>255</td>
<td>48</td>
<td></td>
<td>SRP</td>
<td>75</td>
<td>1.23</td>
<td>292.4</td>
</tr>
</tbody>
</table>

* Beam W19-3 had concrete flange thickness of 56 mm across the whole width of the GFRP box as in Figure 1 and an additional 28 mm thick layer inside the box section, across the full width internally.

All specimens failed through longitudinal cracking at the top corners of the box section starting from the support region and propagating inwards. The initial cracking was sudden and over a wide zone comparable to the support width, with subsequent slow and stable crack propagation towards the mid-span. Before the sudden appearance of the initial crack failure noises of non-visible cracks were heard. No slippage was observed between the UHPC flange and the box beam in any of the specimens. UHPC flanges under the loading region suffered extensive cracking as the corners failed and the longitudinal cracks in the GFRP webs were spread horizontally. Pictures of the failure mode and concrete cracking at the loading region are provided in Figure 2.

3. Longitudinal shear stress assessment

When this mode of failure is examined and thought about it for such loading conditions of high shear forces at the support inducing shear failure, one can say it is a failure related to shear flow at this location; shear flow is higher than the shear strength of this section in direction parallel to the fibre orientation. The shear strength of GFRP in direction of fibres (i.e. longitudinal shear strength) was determined experimentally as 37.9 MPa with a standard deviation of 5.1 MPa. This test is similar to the one described in ASTM D7617/D7917M-11 [7]. This shear strength is determined to be a datum
used in further calculations of shear flow. Shear stress at the upper end of the web was calculated for the maximum load attained by specimens W15-1, W15-2, W19-1, W19-2, W25-1 and W25-2. Specimen W19-3 was excluded due to the irregularities in the cross-sectional dimensions. The values of shear stress at the location of failure are highly different from one specimen to another and could not be correlated specifically to the maximum shear strength of the GFRP in its longitudinal direction. Therefore, the high shear stress at the corners was deemed not be the main reason for failure there.

Figure 2. Mode of failure (left) and concrete cracking (right).

4. Analysis of the reason of failure

A different analysis approach was, therefore, needed to explain the observed failure. A simple 2D non-linear finite element model was used to conduct in-plane analysis of stresses in the cross-section of this hybrid section. The dimensions of specimen W15-1 were used in this model. The model included the UHPC flange and GFRP box section. The UHPC flange was fully connected to the GFRP box section assuming no slippage or separation, using multi-point constraint to completely connect the two surfaces. The bottom CFRP or SRP sheet was not included in the model as it will not affect the 2D behaviour of the cross-section. The lower edge of the GFRP was supported against vertical and horizontal movements. Load was applied to the top of the UHPC flange as a uniform load. 8-node biquadratic plane strain quadrilateral elements were used for both the UHPC and the GFRP, except for the corners where 6-node quadratic plane strain triangular elements were used. The element size for the GFRP was about 2.5 mm except for the corners where the mesh was a bit more dense. An element size of 5 mm was used to model the UHPC component. The solution of the finite element model's equations was performed using the arc length convergence algorithm. The maximum in-plane principal stress at a total applied force of 283 N per mm length of this section is shown in Figure 3.

At this load magnitude, the tensile stresses on the interior part of the corners are higher than the tensile strength of GFRP which is 48.3 MPa. The same tensile stress concentration was obvious at the bottom corners. Based on this finding, the cause of failure was interpreted as a mixed-mode of failure starting as a tensile failure at the interior surfaces of the corners followed by a combination of tensile failure of corners, high shear stresses and outward deformation of the webs of the box sections due to the rotation that develops at the corners.

Deskovic et al. [4, 5] did not observe failure similar to the one presented here. The reason for the difference in the mode of failure is because the GFRP sections in their study were fabricated by winding and consequently, the tensile strength of the corners would be almost the same as the whole section. In contrast, the sections included in this study were unidirectional pultruded sections, so the corners are points of weakness.
5. Conclusions

The shear resistance of these sections could not be directly correlated to the type of bottom CFRP or SRP sheet. The mode of failure of these sections when subject to high shear forces is failure of the corners followed by cracking of the UHPC flange. Traditional calculations of shear stresses at the failure point reveal that high shear stresses are not the only reason for the observed mode of failure. A 2D finite element model was used to interpret the cause of corner failure. Finally, a detailed explanation was presented for the causes of failure in pultruded sections. Thus, a proper reinforcing system should be applied to pultruded FRP sections to ensure the robustness of the corners.

Acknowledgments

The authors acknowledge the contribution of Ms. Sophie Sadowski and the technicians in the department of Civil Engineering of the University of Calgary in preparing and conducting the experimental work. The donations of the materials used in this research by: Fyfe Co. LLC, Lafarge Canada, Hardwire LLC, and Sika Canada Inc. are highly appreciated. Finally, the authors would like to acknowledge the financial support of the Natural Sciences and Engineering Research Council (NSERC).

References