Function-integrated GFRP sandwich roof structure – Experimental validation of design

T. Keller, T. Vallée & J. Murcia

Composite Construction Laboratory, (CCLab), Ecole Polytechnique Fédérale de Lausanne, (EPFL), Station 16, Bâtiment BP, CH-1015 Lausanne, Switzerland

ABSTRACT: A lightweight GFRP roof structure was designed and built in Basel, Switzerland. An experimental program was performed to validate the design of the GFRP sandwich structure of the roof. The program included small-scale tests to validate the material properties of laminates, laminate overlaps and adhesive joints as well as full-scale tests on sandwich beams to validate the design models. For the prediction of the sandwich beam behavior, the same design methods were used as for the roof structure. Material properties from small-scale tests proved to be representative for the beam properties. The beam strength and stiffness responses could be predicted using normal beam theory. The stabilization of the face sheet laminates in compression through the foam core and the through-thickness strength of redirected laminates were underestimated in the design. Both underestimations, however, increased the factor of safety of the roof structure.

1 INTRODUCTION

This paper reports on the experimental validation of the design for the glass fiber-reinforced polymer (GFRP) sandwich roof of the new Novartis Main Gate Building in Basel, Switzerland. The roof concept, dimensions, design and construction are described in a companion paper published in these proceedings (Keller & Haas, 2008). To validate the roof design assumptions regarding material properties of the preliminary design, small-scale experiments were performed on laminate strips, double lap joints and through-thickness coupons. The test results allowed for a decrease in the resistance factors of the final design, according to EuroComp Design Code (Clarke 1996). Furthermore, the design models used for the roof were validated with full-scale sandwich beam experiments. For the beam design, the same design models were applied as for the roof design. This paper describes the experimental work performed on the sandwich beams.

2 EXPERIMENTS ON FULL-SCALE SANDWICH BEAMS

2.1 Beam description

Four full-scale beams were fabricated to experimentally validate the structural design of the roof structure and the applicability of the material strength values from the small-scale tests. To validate the design, the following design checks were made (see also Table 1):
1) Tensile strength of the face sheets,
2) Wrinkling strength of the face sheets and webs in compression (stabilization effect of the foam core),
3) Strength of the epoxy bonded joints between the block-strips (principle: web failure before joint failure),
4) Strength of the laminate redirection points (no delamination of concave laminates),
5) Sandwich stiffness and portions of moment and shear deformations.
The four beams, B1-B4, were built up from eight 900x900x\(d\) mm\(^3\) \((d =\text{beam depth})\) foam blocks, as was planned for the roof (see Fig. 1). The beam length was 7.2 m, the width 0.9 m and the depth 0.3 m with the exception of beam B4, which had a variable depth of 0.3-0.46 m to check the load-carrying capacity of the lower face sheet redirection points (Check 4, see Table 1 and above). Two block-strips from four laminated foam blocks were fabricated and joined with epoxy joints (two vertical adhesive strips of 50 mm width applied on the whole web depth of 300 mm) to form the beam. The adhesive joints of beams B1, B2 and B4 were at mid-span (and therefore not shear loaded), while for beam B3, two joints were placed between the supports and loading jacks to examine the shear strength of the adhesive joints (Check 3 above, see Table 1 and Fig. 1). Subsequently, the upper and lower face sheets were laid-up over the whole beam length.

Table 1. Beam parameters and main results from full-scale beam experiments.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Design check</th>
<th>Foam ([\text{kg/m}^3])</th>
<th>Depth ([\text{mm}])</th>
<th>Joint location</th>
<th>Failure load ([\text{kN}])</th>
<th>Deflection at failure ([\text{mm}])</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>2), 5)</td>
<td>60</td>
<td>300</td>
<td>Mid-span</td>
<td>2x77</td>
<td>136</td>
<td>Lower face sheet below adhesive joint</td>
</tr>
<tr>
<td>B2</td>
<td>1), 2), 5)</td>
<td>145</td>
<td>300</td>
<td>Mid-span</td>
<td>2x71</td>
<td>108</td>
<td>Lower face sheet below adhesive joint</td>
</tr>
<tr>
<td>B3</td>
<td>3), 5)</td>
<td>60</td>
<td>300</td>
<td>Left + right</td>
<td>2x77</td>
<td>149</td>
<td>Web buckling below left jack</td>
</tr>
<tr>
<td>B4</td>
<td>4), 5)</td>
<td>145</td>
<td>300 - 460</td>
<td>Mid-span</td>
<td>2x70</td>
<td>124</td>
<td>Lower face sheet below adhesive joint</td>
</tr>
</tbody>
</table>

Figure 1. Full-scale sandwich beam lay-out and four-point bending set-up beam B1.

The fiber architecture and materials of all beams were identical and corresponded exactly to those of the roof structure. Two types of foam block were used with the lowest and highest densities used in the roof (60 and 145 kg/m\(^3\)) to validate the stabilization effect on the compressed laminate parts (Check 2).

The locations of expected beam failures are summarized in Table 2 (denominated as “failure expected”). Failure in beam B1 was expected to occur in the compressed laminates (upper face sheet or webs) due to the lower core density, while tensile failure of the lower face sheet was expected in beam B2 with higher core density. Beam B3 was designed to fail in the compressed laminates (webs or upper face sheet) and not in the adhesive joints (design principle: web failure before adhesive failure). Beam B4 was designed to fail through delamination within the lower face sheet at one of the concave laminate redirections.
Table 2. Comparison of average properties from small scale tests and corresponding stresses in beams at failure.

<table>
<thead>
<tr>
<th>Component</th>
<th>Material property</th>
<th>Strength from small-scale tests [MPa]</th>
<th>Results from beam experiments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Face sheets in tension</td>
<td>Tensile strength</td>
<td>174.3</td>
<td>171¹, 158¹, 86², 156²</td>
</tr>
<tr>
<td></td>
<td>Through-thickness tensile</td>
<td>11.1</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>strength [MPa]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Face sheets in compression</td>
<td>Compressive strength foam 60 [MPa]</td>
<td>61.3</td>
<td>86¹, -, 86¹</td>
</tr>
<tr>
<td></td>
<td>Compressive strength foam 145 [MPa]</td>
<td>132.5</td>
<td>-</td>
</tr>
<tr>
<td>Webs and adhesive joints</td>
<td>Shear strength, foam 60 [MPa]</td>
<td>30.6</td>
<td>43¹, -, 43¹,², -</td>
</tr>
<tr>
<td></td>
<td>Shear strength, foam 145 [MPa]</td>
<td>49.2</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Shear strength adhesive joint [MPa]</td>
<td>4.0</td>
<td>-</td>
</tr>
</tbody>
</table>

¹ failure expected from design, ² failure in tests

2.2 Set-up and experimental program

A four-point bending configuration was used, as shown in Figures 1. The beams were supported below the transverse double webs that were placed 900 mm from the beam ends, thus resulting in a beam span of 5.40 m. Two steel supports with the same geometry as for the roof supports were used at each support axis (140x35 mm² support surface, support distance 360 mm). The load was applied with two jacks of 300 kN capacity and 150 mm maximum displacement, each jack 1.80 m from the supports with a resulting jack spacing of also 1.80 m. In this way, the loads were introduced into the transverse webs and always one unloaded transverse web was located between the support and the load, as well as between the loads (see Fig. 1). The beams were loaded using displacement-control at rates of 2-8 mm/min in several cycles up to failure.

Figure 2. Beam instrumentation (beam B1).
2.3 Instrumentation

Two load cells were used to measure the loads of the four-point bending configuration. The axial strain distributions through the width of the face sheets were measured at mid-span and mm from mid-span using three gages in each line, and through the webs at 900 mm from the left support using five gages on each beam side (see Fig. 2). On the upper face sheets and webs, strain gages were used (type 6/120LY18 from HBM), while on the lower face sheet omega gages (type PI-2-100 from Tokyo Sokki Kenkyujo, gage length 50 mm) were used for the measurement of larger strains. In total, 16 strain and 6 omega gages were applied for each beam. The vertical deflections were measured with 7 displacement transducers in the beam axis (type K-WA-T-010W/200W from HBM). The arrangement was chosen to determine moment and shear deformations (Check 5 in Table 1).

2.4 Experimental results

All beams showed a similar and almost linear load-deflection response up to brittle and sudden failure. Beams B2 and B4 with higher foam density behaved slightly stiffer than the other beams, while beam B4 was stiffer than beam B2 due to the higher depth between supports and loads. All beams exhibited large deflections at failure (from span/50 to span/36). Table 1 gives the ultimate loads and the mid-span deflections at failure for all four beams.

The loading-reloading paths during the different loads cycles performed for each beam were very similar - no significant visco-elastic behavior was observed. At approximately 40% of the ultimate load, all beams lost some stiffness. At approximately this load level, the adhesive joints at mid-span of beams B1, B2 and B4 started to crack from the lower face sheet on upwards. Ultimate failure in beams B1, B2 and B4 occurred in the lower face sheets just below the adhesive joints at mid-span (see beam B1 in Fig. 3). Figure 3 also shows the crack in the mid-span adhesive joint strip, mentioned above, that propagated up to the upper face sheet at beam failure. The crack initiated and propagated mainly in the web mat layers close to the web-adhesive interfaces on both web sides, thereby crossing the adhesive layer several times. Beam B4 showed slightly different load-deflection behavior during failure. The load could be maintained for about 15 mm of imposed deflection. During this phase, several loud cracks were audible, but no damages were visible from the outside. Beam B3 failed in the web below the left jack through buckling and, immediately afterwards, the upper compressed face sheet also buckled at this cross-section, see Figure 3.

Figure 3. Failure of lower face sheet of beam B1 and failure of web and upper face sheet of beam B3.

The stain distributions through the beam depth always remained nearly linear up to failure, see Figure 4. The strain distributions over the beam width, however, only remained constant up to failure for beams B2 and B4 with the higher foam density (no shear lag effects). The remaining beams with lower density showed approximately 10-15% smaller strain values in the beam axis than over the webs at the ultimate load. However, the differences were still small and at loads corresponding to the serviceability limit state, the distribution remained almost constant.
3 DISCUSSION

The experimental beams were designed according to the same methods as the roof structure. Table 2 gives an overview of the expected beam behavior, based on the mean material property results from the small-scale experiments, and the calculation of the wrinkling and shear strengths according to Eqs. (2) and (3) in the companion paper (Keller&Haas, 2008).

Comparing beam B1 and B2, which differed only in the foam core density, it was expected that in beam B1 (with lower density) the foam core would be too weak to prevent laminate wrinkling, while in beam B2 (with higher density) wrinkling should not occur and that this beam would fail in the lower face sheet, below the adhesive joint at mid-span. Both beams, however, failed in the lower face sheet. It could be concluded that the stiffening effect of the foam core with lower density in beam B1 was underestimated using Eqs. (2) and (3). This underestimation, however, was welcome since it increased the factor of safety of the structure.

Beam B3 failed, as expected, in the webs below one jack through buckling or wrinkling and not in the adhesive joints. The shear stresses in the adhesive joints (2.6 MPa) remained 34% below the mean strength value (4.0 MPa), while the mean strength value in the webs was exceeded by 41% (43 vs. 30.6 MPa). However, the shear stress at failure in the webs (43 MPa) was almost identical to the shear stress in the webs of beam B1 (43 MPa, same foam density), which did not fail in the webs at this stress level. Therefore, it is thought that beam B1 must have also been close to web failure.

Beam B4 was expected to fail by delamination at one of the concave laminate redirections of the lower face sheet. At these points, through-thickness tensile stresses in the laminates of 14.0 MPa were calculated at beam failure. The calculated stress value exceeded the mean strength value (11.1 MPa, see Table 2) by 26% without leading to failure. At this stress level, the lower face sheet failed similarly as observed for beams B1 and B2. Again, a higher factor of safety at these critical locations was a welcome development.

Table 3. Mid-span deflections at 15 kN per jack from measurements and prediction.

<table>
<thead>
<tr>
<th>Beam</th>
<th>From moment [mm] / [%]</th>
<th>From shear [mm] / [%]</th>
<th>Total [mm] / [%]</th>
<th>From moment [mm] / [%]</th>
<th>From shear [mm] / [%]</th>
<th>Total [mm] / [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>26.2 / 87</td>
<td>3.8 / 13</td>
<td>30.0 / 100</td>
<td>17.9 / 76</td>
<td>5.7 / 24</td>
<td>23.6 / 100</td>
</tr>
<tr>
<td>B2</td>
<td>25.6 / 91</td>
<td>2.5 / 9</td>
<td>28.0 / 100</td>
<td>13.7 / 66</td>
<td>7.2 / 34</td>
<td>20.9 / 100</td>
</tr>
<tr>
<td>B3</td>
<td>26.2 / 87</td>
<td>3.8 / 13</td>
<td>30.0 / 100</td>
<td>20.2 / 84</td>
<td>3.9 / 16</td>
<td>24.1 / 100</td>
</tr>
<tr>
<td>B4</td>
<td>23.6 / 93</td>
<td>1.8 / 7</td>
<td>25.4 / 100</td>
<td>12.8 / 68</td>
<td>5.9 / 32</td>
<td>18.5 / 100</td>
</tr>
</tbody>
</table>

From the deflection measurements along the beam axis, the longitudinal shear and elastic moduli were estimated using Timoshenko’s beam theory (including shear deformations). The resulting E-moduli of 17.7-27.4 GPa were considerably higher than the mean elastic tensile
modulus of the face sheets of 12.8 GPa. The values for beams B2/B4 with higher density foam were on average 41% higher than the values for beams B1/B3 with lower density foam. In contrast, the resulting shear moduli of 1.6-2.7 GPa were much lower than the assumed value of 3.0 GPa (based on previous research at the CCLab). However, in similar projects, the shear modulus was also underestimated considerably using this calculation method (Keller and De Castro, 2005), confirming the inaccuracy of this method in estimating elastic and shear moduli and deformations. The resulting high elastic and low shear moduli led to relatively high calculated shear deformations, as shown in Table 3 (16-34% of the total deflection). Calculations with the mean elastic modulus from the small-scale tests for the face sheets \( E = 12.8 \) GPa, \( G = 3.0 \) GPa for the webs, and the \( E \) and \( G \) values for the foams led to shear deformations of 7-13% of the total deflections, which seemed to be more reasonable. However, the total predicted deflections were, on average, 30% higher than the measured deflections (see Table 3). The reason for these variations could be traced back to the varying face sheet and web thicknesses of the beams due to hand lay-up fabrication. Local checks showed an average face sheet and web thickness of 6.5 and 3.9 mm, respectively, instead of the planned 6.0 and 3.0 mm.

The observed cracks in the adhesive joints of beams B1, B2 and B4 initiated at the lower face sheet where the strains in the beam direction were maximum. The ultimate strain of the adhesive joints was therefore smaller than that of the lower face sheets. Since the joints at mid-span did not carry any shear load, this effect influenced neither the beam stiffness nor the strength.

4 CONCLUSIONS

An experimental program was performed to validate the design of a new GFRP sandwich roof structure in Basel, Switzerland. The program included small-scale tests to validate material properties and full-scale tests on sandwich beams to validate the design models. For the beam design, the same design models were applied as for the roof design. Regarding the validation of the design, the experimental results confirmed the following points:

1) The sandwich beam behavior could be modeled using normal beam theory to predict strength and stiffness. The stabilization of the laminates in compression through the foam core and the through-thickness strength of deviated laminates were underestimated in the design. Both underestimations, however, increased the factor of safety of the roof structure.

2) Material properties from small-scale tests were representative for the properties in the beams and characteristic values could be provided as the basis for the final roof design.

3) The experiments confirmed that failure in the sandwich webs occurs before failure in the adhesive web joints.

Based on these results, the beam design was considered to be validated. Since the roof structure was formed and considered as a grid system of orthogonally crossing beams, it was concluded that the design models validated for the experimental beams were also applicable for the roof to predict strength and stiffness.

5 ACKNOWLEDGEMENT

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6 REFERENCES


Keller, T., Haas, Ch. (2008). “Function integrated GFRP sandwich roof structure – Structural concept and design”. Fourth International Conference on FRP Composites in Civil Engineering (CICE2008), Zürich, Switzerland.