POST-BUCKLING SHEAR RESISTANCE OF RIVETED COMPOSITE PANELS

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Abstract
Composite materials are distinguished from other construction materials by their light weight and high resistance. The most common type of composites is glass fibre reinforced polymer (GFRP), due to the facility of element production in pultrusion and the competitive price of glass fibres. The potential of this quite new construction material is far from being exploited, especially for very slender elements loaded in in-plane shear (shear panels). The actual design equations for GFRP shear panels consider the resistance up to elastic buckling only, leading to a low economic competitiveness of such structural elements.

The paper reports on the results of preliminary tests on connections and their potential to serve in GFRP panel-beam connections and a comparison of different models for the buckling behaviour of slender shear panels.

Keywords: blind rivet connections; full-scale tests; glass fibre reinforced polymers (GFRP); non-linear FEM analysis; post-buckling resistance; shear buckling; shear panels

1. Introduction
Current pultruded GFRP profiles usually do not exceed a height of 300 mm. Higher profiles are needed for longer girders in order to attain sufficient stiffness at serviceability limit state. A possible method to produce such higher GFRP girders is the assembly of a Vierendeel frame made of commercial U-profiles braced with GFRP panels. An example of such a Vierendeel beam braced with bolted panels is shown in Figure 1.

Figure 1: Braced Vierendeel Frame of standard profiles and bolted connections
Two main questions emerge for this type of girder: firstly, how the connections of the profiles among each other and to the panels have to be designed and secondly, how to design for buckling resistance of the very slender bracing panels loaded mainly in shear.

Design for shear buckling of webs of plated girders is well known for steel constructions; tension band models are used, relying on the material’s ductility of steel. If such approaches shall be applied to GFRP panels, the missing ductility of FRP must be provided elsewhere in the panel. The goal of the research project presented here is to find out, firstly, if the connections can provide the necessary ductility and secondly, in which way existing tension band models have to be adapted for the design of slender GFRP shear panels.

2. State of the art

2.1 Behaviour of GFRP material

The behaviour of GFRP is usually considered as linear elastic until a brittle failure. Some guidelines are given by Bank [1] for construction applications and Barbero [2] for general applications. More precise information how to design GFRP structure is often provided by manufacturers.

2.2 Buckling of materials

The post-critical behaviour of slender panels can be estimated using several methods. There are two completely different methods with multiple variations. The approach proposed by Höglund [3] is the rotated stress field method, i.e. after the buckling of the panel, only the tension stress increases and the compression stress remains constant: the principal stress directions must rotate in order to maintain equilibrium.

Another approach is proposed by Basler et al. [4], [5] considering tension stresses only. The panel is calculated as a part of a truss beam where the frame carries the bending moment; the panel works as bracing by building a diagonal tension band which guaranties the stability of the system and carries the shear. The difference to the approach from [3] is primarily another definition of the geometry of the tension band. These two methods can be used to design steel panels, but the second method is considering a ductile material behaviour. As GFRP is not to be considered as ductile, it may not be possible to apply this approach.

There also exist design methods for glass panels. Experiments were made by Mocibob [6] and Wellershoff [7]. They adjusted the factors in the rotated stress field method [3] to match with their experiments.

2.2.1 Basler’s model

In Basler’s model [4], the tension band is supported by the frame posts only. The support width is defined by the angle of the tension band to the girder’s axis. The angle has to be half the angle of the panel diagonal. The ultimate shear load after buckling can be calculated using

\[ V_u = V_o + V_{cr} \]

with the post-critical resistance from Eq. (1) and the pre-critical resistance using the linear elastic buckling theory given, for example, in [8].

\[ V_o = \frac{\sigma_t h_f t_w}{2\sqrt{1+\alpha^2}} \]  

(1)

With the post-critical stress \( \sigma_t \) from Eq. (2):

\[ \sigma_t = \sqrt{f_y^2 - \tau_y^2} (3 - (1.5\sin(\theta) ) ) - 1.5\tau_{cr} \sin(\theta) \]  

(2)
Or conservatively from Eq. (3):

\[\sigma_t = f_y (1 - \frac{\tau_{cr}}{\tau_y}) \]  

(3)

A schematic view of the tension band and the angles is shown in Figure 2.

2.2.2 Cardiff's model

In Cardiff’s model [5], Fig. 2, a part of the tension band can also be supported by the frame chord. In order to support a part of the tension band, chords have to be able to create a plastic hinge. As shown in Eq. (4), the bending resistance of those hinges determines the distances \(s_c\) and \(s_t\):

\[s_{c,t} = \frac{2}{\sin(\theta/2)} \sqrt{\frac{M_{pl,N}}{\sigma_{t}t_w}} \leq a\]  

(4)

Eq. (5) allows determining the post-critical shear resistance:

\[V_{\sigma} = \sigma_t h_j t_w \sin(\theta/2)(\frac{1}{\tan(\theta/2)} - \frac{1}{\tan(\theta)}) + \frac{1}{h_j}(s_c + s_t)) \]  

(5)

2.2.3 Höglund’s model

In Höglund’s model [3], which is similar to the one mentioned in the Eurocode 3 [9] and [10], other formulations are described. The ultimate load according to this model is considerably smaller than by the models mentioned above. The equations are omitted in this paper.

2.2.4 Comparison

For comparison reasons (Figure 4), a panel of 1 m x 2 m with a variable thickness has been calculated. The critical buckling loads \(V_{cr}\) are calculated using a buckling factor "k" for pinned edges and clamped edges. In reality, the support conditions are between pinned and clamped. In [3], no buckling factor has to be considered.

There are considerable differences between the models. For a panel with 6 mm thickness, the ultimate load calculated with Basler’s model is about four times higher than according to Höglund’s model. Due to the larger tension bands, the loads calculated with Cardiff’s model are even higher.
Figure 4: Comparison of ultimate shear loads

2.3 Connections

There are several ways to connect GFRP plates to each other. Today, mostly bolted joints are used. Bolted connections are also conceivable, with a larger number of possible failure modes in the composite material to be considered; not to be forgotten is the resistance of the bolt itself. Guidelines are sometimes given by the manufacturer, e.g. in the design manual from Fiberline [11]. In general, the design of bolted connections is similar to bolted steel connections.

Bonded joints are also conceivable they have similar durability and heat resistance as the composite material. Therefore, bonded joints have been tested by several other researchers, e.g. at CCLab [12].

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If there is no possibility to access both sides of the connection, an easy way to connect GFRP plates to profiles are blind rivets. This method is not often used yet; the authors could not find literature about riveted connections in composite applications. The design of riveted connections is similar to bolted connection but failure occurs more often in the rivet than in the composite material.

3. Shear tests on connections

This chapter presents results from shear tests on four different connection means for GFRP panel specimens and GFRP profile specimens.

For each configuration, three specimens are tested. The specimen is fixed between two steel plates using the connection means to be tested at the bottom and bolted connections at the top.
The steel plates are fixed in a hydraulic tension machine. A schematic view of the test set-up is shown in Figure 5.

The slip is measured using LVDTs between the steel plates on side A of the specimen and between the GFRP specimen and the steel plate on side B of the specimen (Figs. 5 to 7, for tests on bolted connections).

![Figure 5: Test set up plan](image1)
![Figure 6: Photo side A](image2)
![Figure 7: Photo side B](image3)

3.1 Bolted connections

First test series focused on M12 INOX A2 bolts, being regarded as standard connection means. The GFRP specimens tested are Fiberline profiles U360 x 108 x 18 with 0°, 90° and 45° orientation of the fibres (specimens R1, R2, R3) as well as isotropic ISOVOLTA Isoval 11 panels with a thickness of 3, 5 and 10 mm (specimens R4, R5, R6). In all the tests R1-R6, two bolts were tested (Figs. 6 and 7). The failure of all bolted specimens occurred in the GFRP plate. Only the bolts on specimen R1 showed some deformation; in all other specimens, the bolts showed no signs of use.

For specimen R1, failure occurred at a load of approx. 125 kN and a slip of 3.5 mm. The failure occurred due to splitting of the laminate in front of the bolt. The behaviour can theoretically be described by a bi-linear relation, with a stiffness change at a load of approx. 40 kN and a slip of 0.1 mm. For specimen R2, failure occurred at a load of approx. 90 kN and a slip of 3.5 mm in the net section of the GFRP plate. A linear behaviour up to failure could be observed. For specimen R3, failure occurred at a load of approx. 100 kN and a slip of 4 mm. Separation of the fibres at an angle of 45° to the load direction caused failure. The test shows a bilinear behaviour up to a load of 20 kN and a slip of 0.1 mm.

For specimen R4, failure occurred at a load around 30 kN and a slip of 14 mm due to splitting in front of the bolt; however, a change of stiffness can be observed at 18 kN and a slip of 1 mm due to lateral pressure failure of the laminate. For specimen R5, failure occurred at a load around 60 kN and a slip of 8 mm. After a laminate compression failure in front of the bolt at 8 mm, failure occurred due to the inclined stress distribution in front of the bolt. The observed behaviour is bi-linear; a change of stiffness can be identified at a load of 40 kN and a slip of 1.5 mm.

Specimen R6 is similar to R1. A part of the results of R5-1 are shown in Figure 8 (left).
3.2 Riveted connection

Aluminium blind rivets with a diameter of 4.8 mm and a grip of 31 – 36 mm were tested on ISOVOLTA Isoval 11 panels with a thickness of 3, 5 and 10 mm (D1, D2, D3). The specimens were 30 mm wide; one single rivet has been tested per specimen.

The test results did not change much from one configuration to another, since the rivet failed in shear. At failure, the load is around 5 kN with a slip of 3 mm. For the thin specimens (D1), one single shear plane formed. For the thicker specimens (D2, D3) two failure planes could be observed.

The observed behaviour is bi-linear, with a stiffness change at a load of 3.5 kN and a slip of 0.5 mm. The results of D2-2 are shown in Figure 8 (middle).

3.3 Glued connections

Two types of glue were tested, both on similar profile specimens as in test series R1-R3: SikaDur 330 at 0°, 90° and 45° (B1, B2, B3) and SikaForce 7710 L35 at 0°, 90° and 45° (B4, B5, B6). The glued area was calculated to resist a load of 60 kN, using the characteristic shear values given by the manufacturer.

The fibre direction does not influence the results considerably. For SikaDur glue, failure occurred due to delamination of the outer layer of the GFRP laminate at approx. 120 kN and a slip of 5 mm. The behaviour is linear elastic until failure. For SikaForce glue, the maximum load is around 20 kN at a slip of 1.7 mm. Failure occurs due to delamination of the top layer of the GFRP laminate. The behaviour is linear elastic until failure, and the variation in the results of each configuration was rather high. The results of B4-3 are shown in Figure 8 (right).

4. Further investigations

4.1 FEM analysis

The next step in this project consists in the development of a non-linear FE model. A single GFRP panel in pure shear shall be analysed. The model will focus on the elastic shear buckling of the panel as well as the modelling of the connection characteristics obtained from tests. Shear resistance depends on the material configuration such as percentage of fibres, layer orientation and matrix type. Therefore, the different producers of GFRP panels often provide characteristics of their products.

The results of the connection tests will be further analysed in order to derive constitutive laws for the connections. Those are then combined to describe the connection between the GFRP panel and the GFRP profile and can be introduced in a FE Model as boundary conditions.
4.2 Full-scale tests

After the determination of appropriate connection means leading to the most favourable buckling behaviour, an experimental verification in full-scale tests on girders braced with GFRP shear panels are intended. The test set-up will be as shown in Fig. 1. The dimensions are to be determined considering the connection means and the FEM analysis results. As an example, such a beam could be used as a balustrade of a pedestrian bridge where it serves as longitudinal static element as well.

4.3 Outlook on modelling of GFRP panels

Nowadays, GFRP panels are considered to have a brittle behaviour, but some connection means may provide a sufficient plastic deformation capacity. As this is exactly what is needed to use tension band approaches for buckling verification, GFRP panels may be designed using an approach similar to Basler’s model [4]. Cardiff’s approach is not appropriate to be applied to this material, because neither the panels nor the profiles are able to provide plastic hinges.

An experimental study on shear buckling behaviour of GFRP plates has been realised at CCLab [13]. The buckling load prediction was made using a numerical approach introduced by Fok [14]. Further studies on other material and geometrical configurations are needed.

5. Conclusions

In this paper, three models for the determination of the buckling behaviour of steel plates where analysed to their applicability for GFRP. Cardiff’s model cannot be used; Wellershoff’s model is currently used for the buckling load determination of GFRP panels, but the low loads lead to the question, if there is a better model applicable to GFRP. The model of Basler can be applied if the material has enough plastic deformation capacity. As the GFRP panel will not provide this plastic deformation capacity, it may be sought in the connection.

In an extensive experimental study it was shown that some connection means do provide a certain plastic deformation capacity. In particular, the following conclusions are drawn:

- Both tested glues showed no ductility. Those glues are therefore no longer analysed in this project.
- The riveted connections showed quite a plastic behaviour. The small load capacity of the rivets can be counteracted by raising the number of rivets; this may also provide a better force introduction in the basic material.
- Some of the bolted connections show a plastic behaviour; however, it is not the bolt that provides the plasticity but the GFRP panel in lateral pressure.
- The panel thickness may contribute to a plastic behaviour in punctual connections such as bolts or rivets. Thinner panels provide plastic deformations; thicker panels are more likely to provoke a brittle failure.

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7. Symbols

\( V_u \) Ultimate shear load
\( \sigma_t \) Post-critical stress
\( S_{cr} \) Support length in the frame chord

\( V_{cr} \) Pre-critical resistance
\( h_f \) Panel height
\( \theta \) Angle of the panel diagonal

\( \tau_y \) Elastic shear stress limit
\( \tau_{cr} \) Critical shear stress
\( \Theta \) Angle of the panel diagonal

\( a \) Panel length
\( t_w \) Panel thickness

\( M_{pl,N} \) Plastic moment in the chords, considering interaction with the normal force

8. References


