ANALYSIS AND DESIGN OF PULTRUDED GFRP PROFILES

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Abstract

The main goal of this work is the structural design and safety checking of GFRP pultruded profiles under compression (columns) and bending (beams). Currently, design codes including safety checking rules for pultruded GFRP structures still do not exist, with the exception of the Italian code (CNR 2008). For the designers and engineers, information is only available in documents published by GFRP manufactures, which often include either contradictory rules or the absence of based rules. The present work describes a set of procedures to evaluate the ultimate strength and deformation of GFRP columns and beams and to check their safety and serviceability. In this work, a design procedure is presented for each type of these members (columns and beams).

Due to its very low elastic modulus in the transverse direction, GFRP profiles are prone to local instability phenomena. In view of this evidence, a study on the strength of GFRP structural components against local buckling is presented. This study aims to compare the (i) analytical predictions, (ii) numerical values and (iii) experimental results of the critical local buckling stress and ultimate stress (at collapse). This comparison makes possible to conclude that GFRP pultruded elements (columns and beams) have substantial post-buckling resistance, which should be taken whenever possible into account in the design.

Finally, the procedures previously presented are applied to the design of a footbridge in GFRP-concrete. A preliminary design of the profiles that support the bridge is performed, as well as the checking for safety in ultimate limit states and deformation in serviceability limit states, according to the proposed methodology. Expectedly, the ELS for deflection govern the design of this type of
structure. Additionally, a check of comfort associated with the frequency of vibration of the footbridge is also performed.

Keywords: GFRP, pultruded profiles, compression, bending, design, deflection, local buckling

1. Introduction

The GFRP is a composite material made of a polymeric matrix reinforced with glass fibers. Due to the longitudinal-disposed glass fibers, GFRP material possesses high strength values (even higher than steel). Despite its high level of strength, GFRP is a very flexible material (elastic modulus of 30 to 40 GPa) when compared with steel. This evidence, associated with the fact that GFRP material creeps when compressed over long periods of time, renders the GFRP behaviour very sensitive to phenomena involving lack of stiffness, for instance, deformation and buckling issues.

It is also an orthotropic material with the fibers oriented in the longitudinal direction of the structural element, which means that its elastic modulus in the transverse direction is much lower than in the longitudinal direction. GFRP also presents a linear elastic behavior until it reaches the rupture load, showing that it possesses a fragile behavior in sharp contrast with steel and other ductile metals. This means the GFRP doesn’t have any ductility, which can be negative, as high deformations before the collapse of the structure serve as a "warning" to people in or around it.

Another distinctive property of GFRP material is its low density. It lowers the self-weight of structures made of it, which makes GFRP a good material to use in bridges, particularly footbridges and in reinforcement of old structures, where inserting steel profiles would cause a high increase in the loading applied to the structure. The low density of this material means also that structures can be assembled very quickly.

2. Design of GFRP columns

The design of GFRP pultruded columns, which have mostly thin-walled sections, is generally governed by the buckling of the element. In longer, more slender columns, buckling is usually governed by global modes like flexural buckling (very common in steel columns) and torsional buckling (only possible in open sections). Shear deformation effects must be accounted for when determining the critical load for any of these modes, as they have some influence in the critical load value. In common design conditions, this influence is usually small (< 5%).

The buckling behavior of shorter (less slender) columns is generally controlled by a local mode in which one of the walls of the cross-section buckles and the other ones, by compatibility, are displaced from their original position. This is known as local buckling mode and Figure 1 shows the deformed configuration of a column with a wide flange I-shaped cross-section.
The determination of the local buckling load of a column (or its local buckling stress $\sigma_{cr,L}$) is not an easy task, due to interaction of the cross-section walls along their longitudinal edges. Kollár (2003) developed a method to calculate the local critical load, which consists of the following simple steps:

- determination of the stiffness components ($D_L, D_T, D_{LT}$ and $D_S$) of the walls that compose the cross-section. If the cross-section is homogeneous, these values are equal for all walls.
- determination of the local critical load for the cross-section with simply supported walls. As represented in figure 2, these walls can be outstand elements or internal elements. An outstand element is a wall that is pinned along one longitudinal edge and free along the other one, like the flanges of an I-shaped section. An interior element is a wall that is pinned along both longitudinal edges, like the web of an I-shaped section and both webs and flanges of a RHS section.
- determination of the elastic restriction to simply supported walls, caused by other walls interacting with them.
- determination of the local critical load and buckling half-wavelength of the column, taking into account the restraining effect from all the cross-section walls (i.e. the elastic restriction given by the interaction between them).

![Internal and outstand elements of an I-shaped cross-section](image)
In certain situations the buckling is not governed neither by a global mode nor a local mode. It is the case of columns with an intermediate slenderness value that buckle in mixed modes caused by interaction between local and global buckling modes. The critical load can be obtained by means of a methodology developed by Barbero and Tomblin (1994), based on the interaction formulae of steel. However, as the behavior of GFRP (linear elastic until rupture) is very dissimilar from the behavior of steel (linear elastic until yielding and then becoming ductile until to rupture), it is questionable to use this method in GFRP column design.

Although the serviceability limit state (SLS) of axial shortening of a GFRP column does not usually governs the design, it can be adopted to obtain a first approach to the size of the profile to use in the referred column. A good way to make the preliminary design is to determine the area of the cross section needed in order to not exceed a limiting value (usually L/1500) for the axial shortening of the column in SLS. The profile obtained with this rule is then checked in ultimate limit states (ULS) of local and global buckling and material rupture.

Note that in the design of GFRP columns and beams to ULS, the material is minored by a safety partial factor $\gamma_m$ that depends on:

- the way how the properties of pultruded material are derived – from test or from laminate theory
- the material being fully or non-fully cured
- the duration of loading
- the glass transition temperature of the material

### 3. Design of GFRP beams

The design of a GFRP beam is usually controlled by the serviceability limit state of deformation. Due to its high resistance and flexibility, the beam reaches the admissible values of deformation for much lower loads than the necessary for collapse or buckling of the element. Table 1 presents the recommended limiting values for deflection of a GFRP beam, according to its use, as referred in Eurocomp (Clarke 1996). In this table, $\delta_{lp}$ represents the long-term deflection and $\delta_{inst}$ the instantaneous deflection.
### Table 1 – Deflection limits adopted in Eurocomp (Clarke 1996)

<table>
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<tr>
<th>Typical conditions</th>
<th>Limits</th>
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<tr>
<td></td>
<td>$\delta_p$</td>
<td>$\delta_p - \delta_{ult}$</td>
</tr>
<tr>
<td>Walkways for occasional non-public access</td>
<td>L 150</td>
<td>L 175</td>
</tr>
<tr>
<td>General public access flooring</td>
<td>L 250</td>
<td>L 300</td>
</tr>
<tr>
<td>Floors and roofs supporting plaster or other brittle finish or non-flexible partitions</td>
<td>L 250</td>
<td>L 300</td>
</tr>
<tr>
<td>Floor supporting columns (unless the deflection has been included in global analysis for ULS)</td>
<td>L 400</td>
<td>L 500</td>
</tr>
<tr>
<td>General non-specific applications</td>
<td>L 175</td>
<td>L 200</td>
</tr>
<tr>
<td>Where the deflection can weaken the appearance of the structure</td>
<td>L 250</td>
<td>-</td>
</tr>
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</table>

The deformation of a GFRP beam due to permanent loading (as well as in a column) is amplified by creep of the material. As in concrete beams, the deflection increases over time periods for uniform loading. Although the SLS of deformation usually governs the design of GFRP beams, it is always necessary to perform a security checking of the ultimate limit states (ULS), namely:

- rupture of material by bending moment
- rupture of material by crushing (due to transverse forces)
- rupture of material by shear
- lateral-torsional buckling
- local buckling
- buckling of the web due to in-plane shear
- buckling of the web due to transverse forces

The local critical load for a GFRP beam can be determined, as seen before for the case of columns, by the method developed by Kollár (2003). In this methodology, the steps are the similar for both flexural and compression members, with some minor differences:

- In a beam, only the compressed flange buckles and the web undergoes deflection by compatibility. Conversely, the bottom tensioned flange restrains the buckling of the cross-section (figure 3).
- The web of a beam is an internal element in bending (in the column case, the web was under compression). This evidence increases the local critical load of the web, thus making it very hard for the web to be the conditioning wall for the beam local buckling.
4. Study on the strength of GFRP structural elements against local buckling

It is known that GFRP profiles are sensitive to local buckling effects. Thus, the aim of this section is to briefly present the results of a study on (i) the use of numerical tools and (ii) the analytical methodology described previously to evaluate the local buckling strength of GFRP structural elements. Three members are studied: one column tested experimentally by Correia (2004), one column tested experimentally and numerically by Turvey and Zhang (2006), and a beam tested experimentally by Bank et al. (1994). Due to space limitations, only the behaviour of the column tested by Correia (2004) is described herein.

Correia (2004) tested an I-shaped GFRP short column with the cross-section represented in figure 4 and 299 mm of length. He obtained an ultimate load of 734.9 KN, corresponding to an ultimate stress of 199.1 MPa. It should be noted that this ultimate stress, although being lower than the compressive strength of the material (375.8 MPa, by an experimental test of Correia, 2004), it is higher than the elastic local critical load, as it will be seen later.
Then, the analytical method by Kollar (2003) was used to estimate the local critical load of the column. The first conclusion was that the local buckling of the cross-section is governed by the buckling of the web. This was expected, as it is a narrow-flange I-shaped cross-section. The application of this method resulted in an estimated local critical load of 455.1 KN, corresponding to an estimated local critical stress of 122.5 MPa, with a local buckling half-wavelength of 122.6 mm.

This column was also analysed numerically by means of a computer program CUFSM (Schaffer 2006), which uses the finite strip method (FSM). The goal of this study was the determination of the local critical load and the buckling half-wavelength of the column. It should be noted that results obtained by the FSM are numerically exact since it accounts for all the interaction effects between walls, making it a good tool to conclude about the validity of results obtained by the analytical method.

From the observation of figure 5, it is seen that the local buckling mode (it always corresponds to the local minimum of the curve) is reached for a critical stress of 118.24 MPa with a half-wavelength of 150 mm. For column lengths higher than 500 mm, the critical load is associated to global buckling (flexural) and it continuously decreases with increasing lengths. For lengths between 150 mm and 500 mm, the buckling of the column should be controlled by local modes with two (or more) half-waves.

Table 2 resumes the local critical stresses ($\sigma_{cr,L}$) and half-wave lengths ($L_{cr}$) obtained experimentally (Correia 2004), analytically (method of Kollár, 2003) and the numerically (FSM analysis using CUFSM, Schaffer 2006).

<table>
<thead>
<tr>
<th></th>
<th>Experimental</th>
<th>Analytical</th>
<th>Numerical</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{cr,L}$ (MPa)</td>
<td>199,1</td>
<td>122,5</td>
<td>118,24</td>
</tr>
<tr>
<td>$L_{cr}$ (mm)</td>
<td>N.A.</td>
<td>122,6</td>
<td>150,0</td>
</tr>
</tbody>
</table>
The value of local critical load estimated by the analytical method is very close to the obtained by the program CUFSM (Schaffer 2006), although a little higher (3.6%). The half-wavelengths obtained from the numerical and analytical methods are very different. This leads to the conclusion that the method of Kollár (2003) is not well tuned to obtain this parameter. More important than that is the fact that the experimental value obtained by Correia (2004) is much higher than any of the two elastic critical stresses (analytical and numerical). The main explanation for this evidence may be due to the post-buckling strength reserve of the column, which is not accounted by the elastic critical stresses. A similar conclusion was drawn from the analyses done for the other two members (column of Turvey and Zhang 2006, beam of Bank et al. 1994).

5. Design of a mixed GFRP-concrete footbridge

This section describes the design of a structural application of GFRP profiles, in particular the design of a footbridge in GFRP-concrete with a concrete slab supported by GFRP profiles. The footbridge has the longitudinal structural model represented in figure 6. The concrete slab has a width of three meters in the transverse direction and a thickness of 100 mm.

Figure 6 – Structural model of the footbridge deck

The preliminary design of the footbridge was made, for SLS, for the admissible limits of deformation referred in table 1, considering the bridge deck with "general public access flooring". A preliminary design approach led to the estimation of three H360 I-shaped profiles (see figure 7). After that, using a more rigorous calculation involving all phases of construction and the creep effects on the GFRP beams and in the concrete slab, it was decided that the final cross-section is composed of five H360 profiles supporting the concrete slab (figure 7).

Figure 7 – Dimensions of a) the cross section of the deck of the footbridge; b) H360 profile (Fiberline 2010)
The safety checking for the ultimate limit state of local buckling was made using the analytical methodology referred in previous sections. However, as the footbridge has a composite section, the GFRP profiles were subjected to bending moment and compression, a case that was not studied in this work. To solve this problem, following conservative assumptions were taken:

- the mid-span section of the profiles, subjected to the maximum positive bending moment, was considered as subject to bending moment only.
- the section of the profile above the supports, subjected to the maximum negative moment, were considered as subject to axial compression only.

The safety checking for lateral-torsional buckling led us to determine the distance between bracing elements of the beams. Using a distance of 4 meters between these bracing elements, the lateral-torsional buckling safety is satisfied.

Finally, a verification of the vibrating frequency of the footbridge to check if people can cross it with comfort was fulfilled (figure 8). The rules in Swiss SIA 160 (1989) and the American Guide Specification (1997) were adopted. To estimate the vibrating frequencies of the first modes of the structure, a model in the program SAP2000 (Computers and Structures 2009) was made, and conclusions were that the bridge has a relatively low frequency for its first vibrating mode.

![Figure 8 - First vibrating mode of the structure: a) 3D view; b) lateral view](image)

6. Conclusions

The GFRP is an orthotropic material, with some specific ways of design. In a beam, because of its high resistance and low rigidity, the design is usually conditioned by the SLS of deformation. For some columns, the design is conditioned by the ULS of local buckling and the resistance determined with the method of Kolár (2003). In longer columns is the ULS of global buckling that governs the design. In a composite GFRP and concrete section, conservative approaches can be made,
considering some sections under uniform compression and other sections only bending, but for a more rigorous design, beam-column formulas should be considered.

References

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