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First-order, buckling and post-buckling behaviour of GFRP pultruded beams. Part 1: Experimental study

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ABSTRACT

Although fibre reinforced polymers exhibit several advantages over traditional materials, their widespread acceptance is being delayed by the lack of appropriate design codes. In fact, additional and comprehensive experimental data are needed to assess the accuracy of recently developed analytical and numerical design tools. This work reports an experimental study on the first-order, buckling and post-buckling behaviours of I-section beams made of GFRP pultruded profiles. Tests were first carried out on small-scale (coupon) specimens, in order to determine the most relevant material mechanical properties. Full-scale tests were then conducted on (i) simply supported beams with spans varying from 1.0 m to 4.0 m under 3-point bending and (ii) cantilevers with spans ranging from 2.0 m to 4.0 m subjected to a tip point load applied at the end cross-section centroid or top/bottom flange mid-point. While the first series is aimed at investigating the flexural behaviour under service and failure conditions (including the local buckling of the top flange), the objective of the second series is to study the collapse behaviour stemming from lateral-torsional buckling. The results obtained confirm that, due to the GFRP low Young's modulus and high strength, the beam structural integrity is often governed by excessive deformation and/or local and global buckling phenomena, rather than by material strength limitations. Moreover, the low shear-to-Young's modulus ratio implies that the role played by the shear deformation is quite relevant, particularly in stocky beams. The experimental data presented here is used to validate and assess the accuracy of numerical simulations reported in a companion paper (Part 2).

1. Introduction

Durability problems experienced by traditional materials, such as steel and reinforced concrete, as well as the need for higher construction speed, have fostered the development of new structural solutions. Fibre reinforced polymer (FRP) materials, in general, and glass fibre reinforced polymer (GFRP) pultruded profiles, in particular, have been playing an increasingly important role in this domain, due to their various advantages over the above traditional materials – *e.g.*, the high strength-to-weight ratio, the low self-weight, the electromagnetic transparency, the fabrication versatility (virtually any cross section shape can be produced), the ease of installation, the low maintenance requirements, the improved durability under aggressive environments or the possibility of being combined with other materials in hybrid structures [1–6].

Currently there are three major drawbacks concerning the use of GFRP pultruded profiles in structural applications: (i) the relatively high production costs, even if they are becoming increasingly competitive, (ii) the behaviour under fire conditions [7–10] and, above all, (iii) the lack of consensual or "official" design codes and guidelines easily accessible and readily applicable by designers. At present, most of the structural usage of GFRP profiles is based on manufacturer design guides/recommendations (mainly presented in a tabular format), which are often incomplete and/or over conservative. Although the *Eurocomp Design Code and Handbook* [11], published in 1996, provides design recommendations for the use of polymer composites in general, this document neither contains a code-like model design basis nor addresses specifically pultruded elements. In 2002, the EN 13706 standard [12] was released – however, this normative document merely defines two classes of materials, associated with minimum values of material properties, and does not provide any design guidance. In 2007, the Italian National Research Council published the first design guidelines for structures made of pultruded profiles [13] – however, these specifications, which are only mandatory in Italy, are still rather incomplete. Before a comprehensive set of design rules and recommendations can be put forward for the use of GFRP pultruded profiles, it is indispensable to carry out research work intended to acquire in-depth understanding on their structural

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behaviour – *e.g.*, their failure modes, namely those triggered by local and/or global buckling phenomena.

The structural behaviour of GFRP pultruded profiles differs significantly from that exhibited by traditional materials. Unlike steel or concrete members, which yield or crack, it may be said that GFRP pultruded profiles exhibit a linear elastic behaviour up to failure, which generally occurs for rather large deformations. In spite of their orthotropic constitutive relationship and layer-wised laminate constitution, GFRP pultruded profiles may be designed on the basis of engineering beam theory, by assuming an equivalent isotropic behaviour [14–17].

Due to both (i) the low Young's modulus and (ii) the significant role played by shear deformation, particularly in stocky members (low slenderness), the design of GFRP pultruded profiles is often governed by deformation constraints [18,19]. Moreover, the low Young's modulus often leads to failures triggered by buckling phenomena, rather than by material strength limitations. Indeed, the collapse of the (globally) slender members is usually associated with global (flexural or flexural-torsional) buckling mechanisms, while the failure of less slender (or laterally braced) members typically stems from the occurrence of local buckling.

The lateral-torsional buckling behaviour of GFRP pultruded I-section beams with narrow (NF) and wide (WF) flanges was experimentally investigated by Mottram [20] and Davalos and Qiao [21], respectively. For this particular buckling phenomenon, the use of the well known equation valid for thin-walled isotropic beams can provide fairly accurate results for standard pultruded profiles, provided that the appropriate elastic constants are employed [22]. Turvey [23] carried out experimental tests on GFRP I-section cantilevers with spans ranging from 0.5 m to 1.5 m subjected to tip point loads. He analysed the effect of the load position on the lateral buckling behaviour and found out that the buckling loads provided by available analytical formulae underestimate the experimentally obtained values by as much as 55% – Turvey attributed these differences to possible inaccuracies in estimating the in-plane shear modulus, pre-buckling deformations and/or other geometrically nonlinear effects. Tests on I-section cantilevers with four different NF and WF cross-sections loaded at the shear centre were reported by Qiao et al. [24] – although these authors obtained a good agreement between the experimental and analytical critical loads (the latter yielded by closed formed solutions developed by themselves), the effect of the load position was never addressed experimentally.

The local buckling behaviour of GFRP pultruded members has also attracted the attention of several researchers, as attested by the number of existing numerical and experimental investigations (*e.g.*, [25–31]). Different analytical and numerical formulations have been proposed to estimate the local critical buckling load. However, as discussed by Mottram [26], there are significant differences between the available experimental results and the predictions yielded by equations either reported in the literature or included in manufacturer manuals. The difficulties in providing accurate analytical estimates of the experimentally obtained local critical buckling loads most likely stem from uncertainties regarding (i) the modelling of the restraints between the different cross-section walls (*e.g.*, at web-flange junctions), (ii) the intrinsic material inhomogeneity and anisotropy and (iii) the unavoidable presence of member mechanical and geometrical initial imperfections. According to Bank [22], a remarkable exception are the closed-formed equations developed by Kollár [31] to assess the buckling behaviour of thin-walled orthotropic sections – they provide results that are in very good agreement with values yielded by both finite element simulations and experimental tests concerning FRP pultruded members. Finally, one last word to mention the GBT (Generalised Beam Theory) formulation that was developed [32,33] and numerically implemented (beam finite elements) [34] by Silvestre and Camotim, which makes it possible to analyse the local and global buckling behaviours of open-section unbranched FRP thin-walled members. Very recently, Silva et al. [35] extended this GBT formulation/implementation to cover FRP thin-walled members with arbitrary open cross-sections and showed that it provides buckling results that compare quite well with numerical and experimental values reported in the literature concerning FRP I-section columns [36,37]. This GBT formulation/implementation is addressed in some detail in the companion (Part 2) paper [38].

The literature review shows that most previous studies on the strength of FRP pultruded members deal with a single phenomenon (either first-order behaviour, local buckling or global buckling). In addition, as discussed in [39], the vast majority of the experimental results reported on full-scale tests concerning structural elements are not accompanied by the appropriate material characterisation (often, no coupon test results are provided), which precludes their use for the validation and calibration of numerical and analytical design tools. However, there is no doubt that the development of widely accepted design standards must be preceded by the acquisition of additional comprehensive and reliable experimental data.

The objective of Part 1 of this two-part paper is to report the results of an experimental investigation carried out at IST and aimed at studying the structural behaviour of GFRP pultruded beams, regarding both serviceability and failure – particular attention is devoted to failure mechanisms triggered by the occurrence of local and global buckling phenomena. A comprehensive experimental campaign was first performed on small-scale specimens (coupons), in order to determine the GFRP profile mechanical properties most relevant for their structural use – this includes tensile, flexural, compressive and interlaminar shear tests performed on GFRP coupons. Full-scale tests were then conducted, in order to analyse the (mostly non-linear) structural behaviour of I-section GFRP pultruded beams at both serviceability and ultimate limit states – in particular, it is possible to identify and characterise the most relevant failure mechanisms. The first test series consists of simply supported beams with spans varying from 1.0 m to 4.0 m, subjected to 3-point bending and loaded with and without lateral bracing – the aim is to investigate the first-order behaviour of GFRP pultruded beams in service conditions and also the failure mechanisms due to lateral-torsional buckling and local buckling of the top flange. The second test series involves cantilever beams with spans ranging from 2.0 m to 4.0 m and acted by a tip point load applied at different depths of the free end cross-section – the purpose is to study the lateral-torsional buckling, post-buckling and collapse behaviour of these GFRP cantilevers.

The companion paper (Part 2) will begin by addressing the numerical modelling of the GFRP pultruded beams, by means of shell and beam finite element analyses – the latter are based on a recently developed Generalised Beam Theory (GBT) formulation. Moreover, it includes a comparison between some of the experimental results presented in this paper, namely those concerning beams whose failure is triggered by local and global (lateral-torsional) buckling, with predictions yielded by the above finite element models and also by available analytical formulae and design equations.

2. Material characterisation tests

The pultruded GFRP I-section profiles considered in the experimental programme, which were produced by the Topglass firm, are made of an isophtalic polyester matrix reinforced with E-glass fibres (inorganic content of 62%, in weight) and exhibit the following nominal dimensions: web height, flange width and wall thickness of equal to 200 mm, 100 mm and 10 mm, respectively. The