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A closed-form equation for the local buckling moment of pultruded FRP I-beams in major-axis bending

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A B S T R A C T

A new closed-form equation for the local instability of pultruded fiber-reinforced plastic beams in bending is derived by substituting suitable buckling approximating functions for compression flange and web into the total potential energy functional. Being obtained from a full-section approach, the equation does not require independent calculations for web and compression flange, which are typical of discrete plate analysis. Moreover, the contribution of the elastic restraint stiffness commonly used to reproduce the web–flange junction behavior naturally arises in the proposed formulation because of the assumed buckling shape. From comparisons with available experiments on 10 beams and FE solutions for 55 beams, the proposed equation appears to be accurate and reliable.

1. Introduction

Pultruded Fiber-Reinforced Plastic (PFRP) thin-walled profiles can be considered, from a macro-mechanical viewpoint, as linearly elastic, homogeneous, and orthotropic, with the axes of orthotropy coinciding with the principal axes of the cross-sections. Their behavior is highly affected by the relatively low values of the Young's modulus, especially in the transverse direction [1], and of the transverse shear elastic modulus [2,3], which more or less coincides with that of the polymeric resin and shows a strong time dependency (see Ref. [4] and references cited herein). Moreover, warping strains play an important role in the mechanical response of composite thin-walled beams, especially in the case of open sections [5]. These features can provoke non-negligible increases in deformations and deflections with respect to isotropic materials and affect both local and global buckling loads. Finally, post-buckling of pultruded shapes is influenced by the strength of web–flange junctions [1,6,7], resin-rich zones from which failure typically propagates [8–10]. As a consequence, PFRP profiles exhibit a complex behavior related to the multi-interaction between shear deformability, non-uniform torsion, and creep, and therefore require suitable modeling criteria.

The flexural-torsional (global) response of PFRP beams has been widely investigated in the literature with regard to both vibrations [11] and buckling [5,12–15]. The present paper, instead, focuses on the local bucking phenomenon, starting from a wide overview of the literature, reported in the next section.

2. Literature review

A brief survey of the literature concerning analytical studies on local buckling of composite structural sections is presented herein. The referenced papers are subdivided into two main categories according to the type of analysis presented.

2.1. Discrete plate analysis

The local buckling analysis of a PFRP shape under axial compression, uniform bending, pure shear, or combinations of the relevant stress states is generally reduced to the analysis of each of the wall segments comprising the shape, which is considered as an individual orthotropic plate that has suitable boundary conditions and is subjected to in-plane loading. In this approach, usually referred to as discrete plate analysis, the longitudinal edges shared by two or more wall segments are usually provided with a continuous elastic restraint, reproducing the stiffening effect due to the adjacent plates.

In Ref. [16], the local instability of carbon-fiber-reinforced

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flanges of I-section beams and columns was analyzed by taking the web restraint coefficient into account.

In Ref. [17], a model for the local buckling analysis of I- and box sections was developed, considering the compression flange as an orthotropic plate that was elastically restrained in correspondence with the web–flange junctions. The rotational spring stiffness was assumed to coincide with the bending stiffness of the web in the transverse direction.

The buckling load of orthotropic plates in uniaxial compression, simply supported along the loaded edges and having one of the unloaded edges elastically restrained and the other free, was found in Ref. [18] by solving the governing characteristic transcendental equation numerically. On the basis of a parametric analysis highlighting the role of the coefficient of elastic restraint, R, a procedure is presented to estimate, from test data on FRP beams, the value of R to be used in the local buckling analysis of the compression flange.

Local buckling of box and I-sections under non-uniform bending was analyzed in Ref. [19]. In particular, it was assumed that bending is mainly resisted by the flanges, which were subjected to constant compression or tension stresses, whereas non-uniform bending stresses on the web panels were ignored. The web panels were instead subjected to in-plane shear. The local buckling of compression flanges and webs, elastically restrained in correspondence with the web–flange junctions, was then evaluated by solving two transcendental equations simultaneously. Simplified expressions for the buckling strengths were finally obtained using a regression analysis.

Explicit expressions for the local buckling strengths of flange and web panels of box and I-section profiles were reported in Refs. [20,21], respectively. In particular, an equation for the local buckling of web panels undergoing non-uniform normal stresses and elastically restrained along the unloaded edges was given in Ref. [20], seemingly for the first time. Other explicit expressions for web and flange panels of different FRP structural shapes were derived in Ref. [22], whereas in Ref. [23] the explicit solution to the eigenvalue problem for a composite plate in uniaxial compression with all four edges elastically restrained was reported, followed by an application to honeycomb sandwich structures.

Currently, the best compromise between accuracy and simplicity, and thus practicality in design, is probably represented by the closed-form local buckling expressions for orthotropic plates derived by Kollár in Ref. [24] by combining the buckling loads of plates without bending stiffness, without torsional stiffness and Huber-orthotropic plates. Following the method outlined by Bleich [25] for steel profiles, these expressions, which take account of the rotational restraint offered by adjacent wall segments, were then applied in Ref. [26] to the local buckling analysis of thin-walled FRP columns and beams.

Kollár's formulation was adopted by the Italian Design Guide CNR DT 205/2007 [27]. With regard to the local flange buckling of I-section beams, an interesting sensitivity analysis of Kollár's equation was presented in Refs. [28,29], where it was shown that this equation correlates significantly better with the experimental results than those that assume that the half-flanges are simply supported in correspondence with the web–flange junction. Comments on the need to take account of the elastic restraint at the web–flange junction and on the advantages of using Kollár's formulation were reported in Ref. [30].

2.2. Analysis of plate assemblies

An approach alternative to that described above consists in applying a variational formulation to the whole thin-walled profile and then minimizing the resulting functional.

Following the work of Bulson [31] on isotropic thin-walled profiles, Zureick and Shih [32] studied the local buckling in FRP beams and columns and deduced the governing stability equations for box and I-section members as special cases. In their proposal, the authors assumed that all plates have the same orthotropic material properties.

In Ref. [33], the case of composite I-sections under pure compression was analyzed with regard to both initial buckling and post-buckling, and a numerical solution to the stability equations was finally developed.

In Ref. [34], the solution to the general characteristic transcendental buckling equation for FRP profiles subjected to eccentric compression was obtained numerically (pure bending was regarded as a particular case). In particular, the actual stress state on the cross-section was approximated by constant and piecewise constant normal stress distributions applied to flange and web panels, respectively. Different properties were considered for web and flanges.

The formulations presented in Refs. [32–34] undoubtedly lead to very accurate reference solutions to the local buckling problem for orthotropic profiles, but they are barely applicable for design purposes. In this context, the development of closed-form expressions would be welcome.

To the authors' knowledge, the only relatively simple closed-form expressions concerning the local buckling of FRP profiles studied as a whole (and not by a discrete plate analysis) are those recently derived in Ref. [35] for box, angle-, I-, and C-shaped sections using the Rayleigh energy method [36]. The closed-form local buckling equation for I-sections presented in Ref. [36] was proposed again in Ref. [37]. These expressions, based on the hypothesis of infinitely long profiles (so as to ignore the influence of the end effects), are restricted to the case of uniform axial compression and assume the same thickness and material properties for all plates comprising the column.

3. Motivation for the study

Kollár's equation [26] is the most widely used expression for the local (flange) buckling resistance of PFRP beams in bending. McCarthy [28] and McCarthy and Bank [29] showed that, in the case of wide-flange I-section beams, the professional bias for Kollár's equation, defined as the ratio of the experimentally determined local buckling strength to the strength predicted by the equation, takes a mean value of 1.20 and exceeds 1.5 for two of the ten profiles investigated (see Table 1, where the reciprocals of this ratio are reported according to a convention more usual in Europe). The test results included in the study were collected from Ref. [38], where the profile stiffnesses obtained from coupon tests were also reported. In the case of columns in pure compression, the professional bias of Kollár's equation is 1.07 [29], indicating that the overestimation of the local buckling strength is influenced by the stress distribution on the web.

3.1. Relationship between local buckling moment and bending moment resistance

The Italian Design Guide [27] recommends that the bending moment resistance of PFRP beams in pure bending be determined as

$$M_{\mathrm{Rd}} = \chi_M(\lambda_M) M_{\mathrm{loc,Rd}} \tag{1}$$

where $M_{\mathrm{loc,Rd}}$ is the design value of the local buckling moment and χ_M (λ_M) is a function of non-dimensional slenderness $\lambda_M = \sqrt{M_{\mathrm{loc,Rd}}/M_{\mathrm{FT,Rd}}}$ (with $M_{\mathrm{FT,Rd}}$ being the design value of the flexural-torsional buckling moment), which accounts for the