Developing an innovative curved-pultruded large-scale GFRP arch beam

TianQiao Liu\textsuperscript{a}, Peng Feng\textsuperscript{a,*, Yuwei Wu\textsuperscript{a}, Shuxin Liao\textsuperscript{b}, Xinmiao Meng\textsuperscript{c}}

\textsuperscript{a}Department of Civil Engineering Tsinghua University, Beijing, China
\textsuperscript{b}Central South University, Changsha, China
\textsuperscript{c}Department of Civil Engineering, Beijing Forestry University, Beijing, China

\textbf{A R T I C L E   I N F O}

\textbf{Keywords:}
FRP composite materials
Curved-Pultrusion technique
Buckling behaviors
Flexural behaviors
Full-scale Pedestrian bridge

\textbf{A B S T R A C T}

An arch glass fiber reinforced polymer (GFRP) I-beam with a 600 mm height was developed based on the latest curved-pultrusion technique to overcome the most critical design issues of large-scale GFRP beams, including excessive deflections and premature buckling failures. To achieve this goal, a review was first conducted regarding the complex buckling behaviors of pultruded GFRP beams, building a theoretical basis for the development of the curved beam. Then, a series of three-point bending tests for beams were conducted at full scale, in which the typical failure modes, load-carrying capacities and deflection and strain data were obtained. Compression flange delamination was found to be the dominant failure mode. The load–strain curves for flange and web plates demonstrated that the proposed beams were exempt from local buckling issues. Additionally, an analytical study and finite element modeling were carried out. Excellent agreement between experimental, analytical and numerical studies was observed. The design approach for conventional straight profiles is readily applicable to curved beams as the difference is limited. In the end, a 20-m-long full-scale GFRP pedestrian bridge was designed, constructed and tested. The great potential of the proposed curved-pultruded GFRP arch beam was successfully demonstrated.

1. Introduction

Pultruded glass fiber reinforced polymer (GFRP) composite materials, with high strength- and stiffness-to-weight ratios, have seen numerous developments in the field of civil engineering in recent decades [28,29,70]. Typical applications of pultruded GFRP profiles include both the main and secondary beams and columns in cooling towers, pedestrian bridges and many types of frame structures [74–76]. In particular, pultruded GFRP beams, due to the relatively low modulus of elasticity and the high anisotropy of the material, tend to exhibit a different behavior as compared to other beams made of conventional materials such as concrete and steel [40,72].

First, pultruded GFRP beams may exhibit excessive deflection under the service limit state, which has substantially limited the possible span length of the beam. Many studies have been conducted to evaluate the flexural behavior of pultruded GFRP beams. Bank [7,8] conducted a series of pioneering tests to investigate the flexural behavior of GFRP I-beams and proposed a test method for determining the flexural and shear moduli. Nagaraj and GangaRao [50], Roberts and Al-Ubaidi [57] and Estep et al. [26] also carried out bending tests on GFRP I- and box-beams, and in their tests, the flexural and shear stiffness were evaluated. Turvey [67] and Turvey and Zhang [69] studied both the major- and minor-axis flexural behavior of GFRP I-beams and proposed design equations to calculate the deflections and flexural modulus. Through all these studies, it has been identified that pultruded GFRP beams tend to fail due to excessive deflection instead of reaching the material strength limit state. Second, a number of studies have also focused on the buckling behavior of pultruded GFRP beams and found that premature buckling failure could significantly reduce the ultimate load-carrying capacities of the beam [15,47,48]). Indeed, pultruded GFRP profiles are commonly manufactured in the form of thin-walled shapes and have a low elastic modulus and a high material anisotropy, making them highly susceptible to complex local and global buckling behaviors. In addition to the excessive deflection and complex buckling behaviors, other types of failure modes have also drawn attention from the field. For instance, both Correia et al. [22] and Zuo et al. [77] have observed the web shear failure of GFRP profiles in their tests studying the flexural behavior of GFRP-concrete hybrid beams.

\textsuperscript{*} Corresponding author.
E-mail addresses: liu_tianqiao@mail.tsinghua.edu.cn (T. Liu), pengpeng@tsinghua.edu.cn (P. Feng), wyw16@mails.tsinghua.edu.cn (Y. Wu), shuxin_l@163.com (S. Liao), mengxinxiao@bjfu.edu.cn (X. Meng).

https://doi.org/10.1016/j.compstruct.2020.113111
Received 10 June 2020; Revised 24 August 2020; Accepted 3 October 2020
Available online 11 October 2020
0263-8223/© 2020 Elsevier Ltd. All rights reserved.
Among all the possible failure modes of pultruded GFRP beams, the deflection and buckling behaviors have been widely recognized as the most critical design factors [22]. In particular, the buckling behaviors may lead to the sudden catastrophic failure of the entire structures and have been emphasized in almost all existing design guides, including those in the United States [1], Europe [27] and China [20]. Thus, the deflection and buckling behaviors of pultruded GFRP beams are of the focus in this work, and a priority is given to the latter. With the critical design issues identified, the main objective of this work is, therefore, to employ the latest curved-pultrusion technique and develop an innovative curved-pultruded GFRP arch beam capable of mitigating adverse deformations as well as preventing premature buckling failures. To accomplish this objective, the latest curved-pultrusion technique was introduced. Next, a review was conducted to acquire a comprehensive understanding of the typical buckling modes, as buckling is considered a prior to other design factors. Then, a new type of curved-pultruded GFRP arch beam was proposed. To investigate the flexural behavior of the proposed beam, three-point bending tests were carried out. Additionally, analytical and numerical studies were conducted to evaluate and validate the flexural behavior of the proposed beam. In the end, a full-scale pedestrian bridge with a span length of 20 m was designed and constructed using the proposed arch beam. This bridge is the first of its type worldwide and has demonstrated great potential for widespread applications of the proposed curved-pultruded GFRP arch beam.

2. Curved-Pultrusion Technique

In common practice of bridges, a camber is often introduced to steel and concrete beams/girders to mitigate the adverse deflection and to ensure the clearance under bridge. However, the conventional pultrusion technique, namely the straight-pultrusion process, can only produce straight profiles [58], as shown in Fig. 1a. In this case, the camber is not practically achievable, and consequently, a large deflection is often inevitable. One common approach of reducing the vertical deflection is to increase the section depth of GFRP beams, which would, however, lead to an increased cost of the material. In this regard, the latest curved-pultrusion technique was adopted to realize the arch/cambered GFRP beam, as shown in Fig. 1b. In the curved-pultrusion process (also referred to as the bent- or radius-pultrusion), the GFRP section coming out of the forming and curing die is further bent to a desired curvature at the pulling and curing system before the resin is completely hardened [18,63]. Typically, only circular curvature is permitted. The curved-pultruded beam, namely the arch/cambered beam, permits an offset to the adverse vertical deflection, thus ensuring the required clearance under the beam as well as satisfying the service limit state of the structure.

It is noted that to achieve the curvature of pultruded profiles, ultraviolet (UV) radiation can also be used. An early study on the use of UV radiation to produce curved-pultruded profiles was conducted by Kényedi and Kusy [36], in which a 0.5-mm-diameter round quartz fiber reinforced composite profile was successfully produced. Then, Britnell et al. [18] adopted UV radiation instead of mechanical curving system in pultruded process and produced a variety of structural shapes such as the solid circular bar, and angle and box sections. Additionally, Tena et al. [62] further investigated the effect of emitting intensity of UV radiation and the pulling speed on the curved-pultrusion process. Nonetheless, using UV radiation the geometries of curved-pultruded profiles are often limited, such as those having 6 mm in diameter (the circular bar described in [18]) and 10 mm in width (the rectangular section described in [62]). Such a small geometry can hardly be used in typical civil infrastructures. In this regard, the curved-pultrusion technique using mechanical curving system is focused in this work. Recently, using such curved-pultrusion technique Tonatto et al. [64] produced a spiral beam having a radius of curvature of 581 mm and a length of 335 mm, and its flexural behavior was studied. They reported that the curved-pultrusion technique, as compared with the filament winding process, could yield a more homogeneous fiber distribution, thus permitting an increased flexural strength. In the present work, a large-scale curved-pultruded I-beam was studied. The section height of this beam is 600 mm, which is, by far, the largest in the available literature.

3. Review of buckling modes of pultruded GFRP beam

Excessive deflection and buckling behaviors have been identified by many researchers as the most critical design factors for GFRP structures (Correia et al. [22], for instance). In particular, the buckling behavior may lead to catastrophic failure of the entire structure and thus, is given a precedence in this work. Representative studies have been systematically reviewed to acquire a comprehensive understanding of the buckling behaviors. It is noted that this review does not indicate that the buckling failure is the only failure mode of pultruded GFRP beams. Other failure modes pertaining to strength limit state should be prevented through appropriate detailing. Typically, three types of buckling modes are of the highest interest in the field, including 1) flange local buckling (FLB) and/or web local buckling (WLB); 2) lateral torsional buckling (LTB); and 3) local section distortion (LSD) affected-LTB, as shown in Fig. 2. In particular, FLB and WLB often occur simultaneously as flange distortion could lead to the rotation of the web. Nonetheless, the flange plate is restrained only at one end and the web plate is restrained at two ends, and the width of flange plate is often not greater than that of web plate; thus, FLB strength is often much higher than WLB strength for a beam having the same flange and web thickness. In this regard, WLB due to flexural behavior of the beam is typically not a critical design concern and thus, is not specifically reviewed in this work. However, the web crippling and web compression buckling (WCB) triggered by concentrated loads are discussed.

3.1. Flange local buckling

Flange local buckling (FLB) (see Fig. 2a) often occurs in thin-walled flexural members in which the flange plate is essentially subjected to compression and tends to buckle prior to reaching the material strength limit state. The earliest study investigating the FLB behavior of pultruded GFRP profiles dates back to the 1990s when Barbero et al. [15] conducted experimental tests on GFRP I- and box-beams and first observed the local buckling of the compression flange (see Fig. 3a). In their study, classic plate theory was used and the compression flange was assumed to be a simply-supported plate with one edge free and the other restrained by the web plate via an elastic spring with rotational stiffness k, as shown in Fig. 3b. Barbero and Raftoyiannis [13] continued their work on predictive models by taking flange-web junctions into consideration. Later, Bank et al. [11] highlighted, again, the impact of considering the restraining effect of flange-web junctions in the predictions of the local buckling loads. Bank et al. [10] also carried out bending tests on GFRP I-beams and identified that local buckling could trigger the complete failure of the beam. Then, Pecce and Cosenza [53], Qiao et al. [55] and Qiao and Zou [54] proposed design equations to predict the critical FLB stress and similar to previous studies, the flange plate was assumed to be elastically restrained at the flange-web junction.

In particular, Kollár [37] proposed a full suite of explicit design equations calculating the critical buckling loads for orthotropic plates with various types of boundary conditions. Kollár’s equations are generally taken as the benchmark solutions for the field and have been adopted in American [1], European [27] and Chinese [20] design guides. Correia et al. [23] also carried out bending tests on GFRP I-beams and again confirmed that pultruded GFRP profiles were gov-
erned by buckling and deformation criteria instead of material strength criteria. In addition, other researchers [2,3] also proposed closed-form equations to predict the FLB load of GFRP beams, in which the shear deformation and flange-web junction were considered.

Recently, Vieira et al. [71] extensively augmented the data by conducting 62 four-point bending tests on pultruded GFRP I-beams with varying flange slenderness ratios b/2tf (ranging from 6 to 12) and span configurations (ranging from 1800 to 2900 mm). Based on the test results, they also derived an explicit solution for predicting the critical FLB strength of GFRP I-beams [43]. In the derivation of that equation, the classic plate theory was used, and more importantly, a coefficient of 2, determined via regression with respect to the finite strip method, was introduced to the rotational restraint of the flange plate proposed by Kollár [37], and consequently, an improved prediction of the FLB strength of GFRP I-beam was obtained. Following the test method by Vieira et al. [71], Liu and Harries [41] carried out 10 bending tests on GFRP box-beams. Kollár’s equation, though commonly known to provide conservative solutions, was found to overestimate the critical FLB loads by up to 41% for the box-beams being addressed. This was due to the poor fiber-matrix architecture observed at the flange-web junctions. In this case, the rotational stiffness of the flange-web junction \( k_j \) (see Fig. 3c) should be considered, since in classic FLB solutions [15,37], the flange-web junction was assumed to be rigid (\( k_j = \infty \)) and only the flexural stiffness of web \( k_w \) was used to represent the overall restraint \( k \) to the flange plate.

In the above studies, the flange-web junction has been reported to have a substantial influence on the as-observed FLB behavior of pultruded GFRP beams [9,10,43]. Additionally, the pioneering studies by Barbero and Rautoyiannis [13] and Mosallam and Bank [48] have pointed out that the flange-web junction of pultruded GFRP profiles is not rigid, opposing to the classic assumption in all commonly accepted design equations. In this regard, Turvey and Zhang [68], Mosallam et al. [79] and Xin et al. [78] conducted experimental tests to quantify the rotational stiffness of the flange-web junction \( k_j \). Recently, Liu and Harries [41] and Liu et al. [44] adopted the test fixture proposed by Turvey and Zhang [68] and evaluated \( k_j \) for box- and I-sections, respectively. From all these test results, overestimated elastic restraints to flange plates were observed. It may be argued that when using the specified test fixtures, the flange-web junctions made of other type of material (steel, for instance) could also appear to be non-rigid. To this argument, it must be noted that the assumption of rigid flange-web junctions in hot-rolled or cold-formed steel shapes has been sufficiently validated through countless applications. As for the pultruded GFRP profiles, the fiber-matrix architecture is not uniform across the section (this is almost inevitable due to state-of-the-practice of pultrusion process); that is, the flange-web junction, at the material level, may differ from the other parts of the flange plate. According to an ongoing study by the present authors, the fiber volume ratio \( V_f \) at the flange-web junction region may be less than that of the other parts of a flange plate, while the mechanical properties
used in calculating the rotational stiffness as well as the critical FLB strength are those measured at the flange plate (in fact, no standard coupon can be extracted from flange-web junction region). With the actual $k_i$ considered, the predicted FLB strength could be improved, particularly for beams having a poor fiber-matrix architecture at the flange-web junctions [41].

### 3.2. Web crippling and compression buckling

In addition to the local buckling of flange plates, Borowicz and Bank [16,17] investigated the possible failure modes of web plates. Borowicz and Bank [16] first tested 20 GFRP I-beams subjected to concentrated loads and identified interlaminar shear strength-dominated web crippling failure, which was characterized by shear failure at the flange-web junction. The corresponding design equation was developed and has been adopted by ASCE Prestandard [1]. In particular, the bearing plate was found to provide an additional capacity to the beam with regard to web crippling failure. Then, Borowicz and Bank [17] tested five deep GFRP I-beams subjected to concentrated loads and observed a web buckling-dominated failure mode. Differing from the beams that failed due to the material strength-dominated web crippling failure, the bearing plate showed little impact on the ultimate capacity of the beam failed due to web compression buckling, which is the stability-dominated failure mode. Kollár’s equation was, again, reported to greatly underestimate the web buckling strength, approximately by a factor of 2.

#### 3.3. Lateral torsional buckling

Lateral torsional buckling (LTB) (see Fig. 2b) is a typical design concern, particularly when the compression flange is only intermittently or minimally laterally supported, such as those flexural members in cooling towers and truss bridges. To investigate the LTB behavior of pultruded GFRP profiles, a number of studies have been conducted. Mottram [49] conducted experimental tests on GFRP I-beams, in which the LTB behavior was observed and found to be extremely sensitive to imperfections in the test setup. Later, Brooks and Turvey [19] used a different test configuration, cantilever bending test, to study the LTB behavior of GFRP I-beams. Pandey et al. [52] extended the flexural tests on GFRP I-beams by using different load patterns. In addition, Turvey [65,66] carried out tests on cantilever box- and I-beams. A design formula, modified from that for isotropic material, was found to have a good agreement for those beams having relatively high span-to-depth ratios. In a numerical study, Lin et al. [39] constructed a finite element (FE) model to investigate the LTB behavior of GFRP beams and shear deformation was found to have a more prominent impact on the orthotropic GFRP materials when compared to the isotropic materials.

In addition, Davalos et al. [25] developed an analytical solution based on the principle of the minimum potential energy. Sapkás and Kollár [59] also developed explicit design equations to predict the LTB loads of GFRP beams and various load patterns were considered. Later, Qiao et al. [56] and Correia et al. [23] conducted flexural tests on cantilever GFRP I-beams and developed analytical solutions to predict the LTB loads. Once again, it was confirmed that the pultruded GFRP beams were essentially governed by deformation criteria rather than material strength criteria [23]. In addition, Ascione et al. [4] proposed a numerical model for predicting the LTB loads of GFRP I-beams. Nguyen et al. [51] experimentally evaluated the effect of load positions on LTB behavior of GFRP I- and Channel-beams; for instance, the beams loaded at the centroid exhibited LTB loads approximately
40% higher than those loaded at the top flange. Recently, Vieira et al. [71] conducted 86 three-point bending tests on GFRP I-beams, in which five different I-sections were loaded under five span configurations, resulting in a total of 25 different lateral slenderness ratios \( L_b/\tau_y \), ranging from 42 to 301 (\( \tau_y \) is the radius of gyration about the y-axis). Based on the test results, they also derived an explicit design equation for predicting the critical LTB loads of GFRP I-beams subjected to various types of load patterns [42]. Similar to the findings from [64], this design equation was only able to provide accurate solutions for those beams with relatively high lateral slenderness ratios (\( L_b/\tau_y > 100 \), for instance), while for those beams with lower lateral slenderness ratios (\( L_b/\tau_y < 100 \), for instance), local section distortion (LSD) was observed and found to affect the buckling strength. In addition, Zeinali et al. [73] experimentally and numerically evaluated the LTB of pultruded GFRP I-beams subjected to pure bending, and the design equation for steel sections was found to be valid for slender GFRP beams.

3.4. Local section distortion affected lateral torsional buckling

As aforementioned, lateral torsional buckling is typically the dominant behavior for GFRP beams with high lateral slenderness ratios [42,64], whereas for GFRP beams with intermediate lateral slenderness ratios, various extents of the flange and/or web local buckling/deformation may occur accompanying the global LTB behavior, substantially reducing the buckling strength of the beam. This type of buckling mode may be referred to as interactive buckling [35] and coupled lateral and distortional buckling [14]. Based on the practical experimental observations by Vieira et al. [71], LTB essentially dominates the global behavior of the entire beam, while flange and/or web local buckling/deformation only occurs within a limited span of the beam—under the concentrated load; that is, the interaction between global and local buckling was not seen throughout the beam length. Thus, the term of local section distortion affected lateral torsional buckling, namely LSD affected-LTB (see Fig. 2c), was proposed by Liu et al. [42]. In fact, the numerically observed interaction between global and local buckling modes, such as those presented by Kabir and Sherbourne [35] and Laudiero et al. [38], can hardly be seen in the practice of pultruded GFRP flexural members subjected to common loading patterns (three-point bending, for instance), though they are often observed in steel beams [80]. This effect is due to the fact that the GFRP material typically behaves in a linear manner up to failure without showing any plasticity similar to that seen in steel members. Additionally, during the loading process of GFRP beams, either type of global or local buckling could trigger the sudden brittle failure of the entire beam, thus prohibiting the excessive development of local buckling/deformation in the existence of global buckling. Both the tests by Insausti et al. [32] and Liu et al. [42] have demonstrated that only the limited beam sections under concentrated loads were observed to be locally distorted, while the rest of the beams only exhibited LTB behaviors, as schematically shown in Fig. 2c. With this buckling mode defined, Barbero and Rafayounian [14], Davalos and Qiao [24], Insausti et al. [32] and Liu et al. [42] investigated LSD affected-LTB behavior, and the corresponding design equations were proposed. In particular, Liu et al. [42] provided a practical design suggestion advising the selection between LTB and LSD affected-LTB equations for GFRP I-beams with flange slenderness ratios \( b/2\tau_y \) from 4 to 12 and lateral slenderness ratios \( L_b/\tau_y \) from 42 to 301. On the other hand, Kabir and Sherbourne [35] proposed design equations for interactive buckling, namely the actual interaction between global and local buckling throughout the beam length. In their equation, a linear combination of global and local buckling modes was considered. Laudiero et al. [38] also analyzed the interactive buckling mode through FE analysis and confirmed that this buckling mode was prominent for beams with an intermediate slenderness.

3.5. Summary

Based on this review, it can be concluded that the main factors affecting the buckling behaviors of pultruded GFRP beams are the flange slenderness ratio \( b/2\tau_y \) (for FLB), web slenderness ratio \( d/\tau_y \) (for WLB, crippling and compression buckling) and lateral slenderness ratio \( L_b/\tau_y \) (for LTB). In particular, the lateral slenderness ratio, together with the flange slenderness ratio, could determine the LSD affected-LTB. With the buckling behaviors understood, in this work, a new type of GFRP I-beam was proposed to prevent premature buckling failures, thus permitting the improved flexural strength of GFRP structures.

4. Experimental program

4.1. Materials and specimens

With the curved-pultrusion technique, the arch/cambered beam was realized in order to mitigate the adverse deflection. Additionally, with the buckling behaviors determined, the optimal section geometries were designed to avoid buckling failures. Thus, a new type of curved-pultruded GFRP arch beam was proposed by the present authors and was produced by Beijing Golden Bridge (Composites) Tech Co., LTD., China. The GFRP materials consisted of E-glass fiber (supplied by Taishan Fiberglass Inc., China) and epoxy resins (supplied by South Asia Epoxy Inc., China), and the fiber weight ratio was 72.5%, which was measured in a burn-off test [33], as shown in Fig. 4. The pulling speed of curved-pultrusion process was 0.07 m/min. The detailed specifications regarding the pultrusion process are referred to the manufacturer; in this work, only the structural behavior of the curved-pultruded beam is focused. A total of three I-beams were produced, including one with a span length of 7 m, denoted as SS-7, and two with span length of 12 m, denoted as SS-12A and SS-12B. Detailed beam geometries are shown in Table 1 and Fig. 5. In Fig. 5, it is shown that each beam consisted of three I-sections with geometries of \( 600 \times 218 \times 10 \times 15 \) mm (depth \( \times \) width \( \times \) flange thickness \( \times \) web thickness), and the corresponding beam assemblage scheme is shown in Fig. 5b. In particular, the geometry of the beam was designed for a practical project—a 20-m-long pedestrian arch bridge, as shown in Fig. 15, and the objective of this design was to achieve the strength limit state of the material and avoid all possible premature failures such as buckling issues. All curved I-sections were continuously manufactured from one batch of material and cut into the desired length; thus, they all had an identical radius of curvature. Steel bolts and epoxy adhesives were used to combine the I-sections. Stainless steel bolts were used so as to ensure a uniform corrosion resistance. With three I-sections combined, the rotational stiffness of beams SS-7 and SS-12 was greatly increased, avoiding the possible LTB failure. It is noted that the feasibility of assembling multiple beams using steel bolt/rebar has been successfully demonstrated by Liu et al. [46] in a hybrid multicell GFRP-concrete beam. Additionally, the outstanding flange plates were bonded in the interior cells, eliminating the free ends of the flange plates and substantially improving the FLB strength. The web plates were also designed to have a thickness of 15 mm so that all the web-related buckling behaviors can be prevented. In addition, following the method by Liu et al. [46], wood and FRP blocks were installed at the end sections over a span of 300 mm to prevent local bearing failure due to concentrated loads at the supports. In conclusion, the proposed beam section was designed to prevent possible premature failures due to buckling issues.

In addition, material characterization tests were conducted to measure the mechanical properties of the GFRP materials addressed in this work. Test results are presented in Table 2. In particular, the longitudinal tensile strength \( F_{ti} \) and modulus \( E_{ti} \) were measured from the flange plate [5]; the transverse tensile strength \( F_{t\perp} \) and modulus \( E_{t\perp} \)
were measured from the web plate [5]; the in-plane shear strength $G_{LT}$ and modulus $F_{LT}$ were measured from the web plate [6]; the longitudinal compressive strength was measured in the full-section compression test [31]; and the interlaminar shear strength $F_{sh,int}$ and modulus $G_{sh,int}$ were measured from the web plate [30]. The web plate can be cut into straight coupons, and thus, the standard test methods are readily applicable. Nonetheless, the flange plate is curved, and straight coupons cannot be obtained. The radius of curvature of the flange plate is 56,006 mm, and correspondingly, the longitudinal tension coupons with a span length of 235 mm would yield an out-of-straightness of approximately 0.2 mm, which is less than 1/1000 of the coupon.

**Table 1**

<table>
<thead>
<tr>
<th>Beam geometries (mm)</th>
<th>SS-7</th>
<th>SS-12A</th>
<th>SS-12B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam height $d$</td>
<td>600</td>
<td>600</td>
<td>600</td>
</tr>
<tr>
<td>Beam flange width $b$</td>
<td>618</td>
<td>618</td>
<td>618</td>
</tr>
<tr>
<td>Flange thickness $t_f$</td>
<td>10</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Web thickness $t_w$</td>
<td>15</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>Total span $L$</td>
<td>7000</td>
<td>12,000</td>
<td>12,000</td>
</tr>
<tr>
<td>Clear span $S$</td>
<td>6700</td>
<td>11,700</td>
<td>11,700</td>
</tr>
<tr>
<td>Arch rise $S$</td>
<td>109</td>
<td>322</td>
<td>322</td>
</tr>
<tr>
<td>Radius of curvature $R$</td>
<td>56,006</td>
<td>56,006</td>
<td>56,006</td>
</tr>
</tbody>
</table>

Fig. 4. Burn-off test of GFRP material.

**Fig. 5.** Beam geometries and assemblage (unit: mm).
length. Given such a small out-of-straightness, it was determined to also use the standard test method for curved flange coupons. In addition, when compared to the minimum values prescribed by design guides, the GFRP material used in this work showed an exceptionally high tensile strength and modulus, and the compressive and shear properties were also greater than the minimum values.

### 4.2. Test setups and instrumentation

To investigate the flexural behavior of the proposed curved-pultruded GFRP arch beams, three-point bending tests were conducted. The test setups of SS-7 and SS-12A are presented in Fig. 6a and 6b, respectively. SS-12B and SS-12A were essentially the same beam, and thus, the test setup of SS-12B was identical to that of SS-12A. All three beams, SS-7, SS-12A and SS-12B, were simply-supported on the steel hinge and roller. As mentioned above, the end sections were strengthened using wood and FRP blocks to prevent premature local bearing failure at the supports, as shown in Fig. 6c. A hydraulic actuator with a capacity of 3000 kN was used to apply the concentrated load \( P \) at the mid-span. In each beam, 15 electrical-resistance strain gages were installed on the web at mid- and quarter-spans (five for each location) to monitor the possible WLB during the tests, and nine additional strain gages were placed on the top flange near mid-span and at quarter-spans (three for each location) to monitor the possible FLB. In particular, the strain gages on the top flange were uniformly placed across the entire flange, thus enabling the monitor of both the interior and exterior flanges. Three linear variable differential transducers (LVDT) were installed under the beam at mid- and quarter-spans (one for each location) to measure the vertical deflection. The tests were conducted in the manner of displacement control. All the beams were loaded to failure, during which the loads, deflections and strains were continuously recorded.

### 4.3. Results and discussion

The measured load-deflection curves and the ultimate failure modes of all three beams are shown in Fig. 7. It is evident that the curved-pultruded GFRP arch beams behaved in a linear manner until failure. The ultimate loads \( P \) of SS-7, SS-12A and SS-12B are 900, 599 and 566 kN, respectively, and the moment capacities of those beams are 1509, 1759 and 1662 kNm, respectively. It is noted that in the calculations of moment capacities, the self-weight of the beam, 34 kg/m, was considered. In general, the proposed beams all failed by reaching the material strength limit state. In addition, the detailed failure modes of each beam are presented in Fig. 8. The failure mode of SS-7 was characterized as flange-web junction shear failure, as shown in Fig. 8a. When reaching the ultimate moment of 1509 kNm, SS-7 suddenly lost its load-carrying capacity, indicating brittle material strength failure. As for SS-12A, the compression flange near the load point first delaminated, as shown in Fig. 8b. Then, flange delamination triggered the shear failure of the epoxy adhesive, leading to the separation of the I-sections. Similar to SS-7, SS-12A showed a brittle failure when reaching the ultimate moment of 1759 kNm. Despite of showing a sudden failure, compression flange delamination was clearly observed to initiate the failure of the entire beam. In addition, SS-12B failed at 1662 kNm. The compression flange delamination was, again, observed near the load point, substantially reducing the capacity of the top flange, as shown in Fig. 8c. Then, flange-web junction and epoxy adhesive shear failures were observed. Moreover, the damage propagated through the height of the beam, rapidly spreading across the web plate and reaching the bottom flange; that is, in Fig. 8c, the web plate was buckled, and the bottom flange-web junction was cracked.

As aforementioned, the excessive deflection has been recognized as one of the main issues limiting the applications of pultruded GFRP beams. In this work, three curved-pultruded GFRP arch beams, SS-7, SS-12A and SS-12B, were designed to have initial cambers of 109, 322 and 322 mm, respectively. At their ultimate failures, the measured mid-span deflections of SS-7, SS-12A and SS-12B are 69, 235 and 199 mm, respectively. That is, all the deflections can be completely offset by the provided cambers. In this regard, the use of curved-pultruded GFRP arch beam in mitigating the adverse deflection is proven. On the other hand, no obvious buckling was seen in any one of the three beams before their ultimate failures. The buckling behaviors of the three beams were further investigated by reviewing the strain readings in the following sections.

The strain changes in the flange and web plates of each beam at mid- and quarter-spans were recorded throughout the tests. First, the plane-section assumption was confirmed. Taking SS-12B as an example, Fig. 9 shows the strain distributions across the entire height of the beam at four load levels (\( \mu \) indicates the ultimate bending moment of the beam); for the strains at mid-span, no strain gage was placed at the top flange under the load point. In general, the strain distributions at both mid- and quarter-spans were almost linear across the beam height. Thus, it is concluded that the plane-section assumption is valid for curved-pultruded arch beams, providing a theoretical basis for further analytical and numerical studies.

Second, the measured load-web strain curves at mid-spans of SS-7 and SS-12A are presented in Fig. 10. The web strain of SS-12B is not shown, as it exhibited almost the same behavior as SS-12A. In Fig. 10, two strains, symmetrically measured at the compression (510 mm) and tension zones (90 mm) of the web, are shown (only one I-beam, instead of the assemblage of three beams, is shown in Fig. 10 for simplicity purpose). It can be seen that both the strains at

### Table 2

<table>
<thead>
<tr>
<th>Material properties (MPa)</th>
<th>Experimental results (MPa) (COV)</th>
<th>Coupon geometries’ (mm)</th>
<th>Test methods</th>
<th>Minimum values by design guides (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( E_t )</td>
<td>768 (0.04)</td>
<td>235 × 10 × 10 (flange)</td>
<td>ASTM D3039</td>
<td>400</td>
</tr>
<tr>
<td>( E_l )</td>
<td>49.160 (0.03)</td>
<td></td>
<td>ASTM D3039</td>
<td>30,000</td>
</tr>
<tr>
<td>( F_{lt} )</td>
<td>61.6 (0.08)</td>
<td>235 × 10 × 15 (web)</td>
<td></td>
<td>45</td>
</tr>
<tr>
<td>( F_{lt} )</td>
<td>12,607 (0.06)</td>
<td></td>
<td></td>
<td>7000</td>
</tr>
<tr>
<td>( L_{fl} )</td>
<td>229 (0.07)</td>
<td>Full-section compression</td>
<td>GB/T 31,539</td>
<td>7000</td>
</tr>
<tr>
<td>( G_{fl} )</td>
<td>3411 (0.07)</td>
<td>320 × 15 × 15 (web)</td>
<td>ASTM D5379</td>
<td>4000</td>
</tr>
<tr>
<td>( V_{fl} )</td>
<td>71.3 (0.13)</td>
<td></td>
<td></td>
<td>552</td>
</tr>
<tr>
<td>( G_{sh,fl} )</td>
<td>3162 (0.08)</td>
<td>35 × 15 × 15 (web)</td>
<td>GB/T 1450.1</td>
<td>28</td>
</tr>
<tr>
<td>( \mu_{fl} )</td>
<td>69.8 (0.08)</td>
<td></td>
<td></td>
<td>24</td>
</tr>
</tbody>
</table>

\( FLt \) indicates failure. The ultimate loads \( P \) of SS-7, SS-12A and SS-12B are 900, 599 and 566 kN, respectively, and the moment capacities of those beams are 1509, 1759 and 1662 kNm, respectively. It is noted that in the calculations of moment capacities, the self-weight of the beam, 34 kg/m, was considered.
Fig. 6. Three-point bending test setups.

a) SS-7

b) SS-12A

c) End-section details
the compression and tension zones of the web are all linear up to the failure. Thus, it can be concluded that no web-related buckling, namely WLB, crippling and compression buckling, occurred in the tests.

Finally, the load-top flange strain relationships measured near the mid-span of each beam are shown in Fig. 11. For each beam, the strains at both the interior and exterior flanges were almost the same (thus, in Fig. 11 the interior and exterior strains are not specifically labeled), and both strains increased linearly with the increasing load, indicating that no FLB occurred during the tests. In particular, the interior flange strain of SS-7 (yellow line in Fig. 11) showed a turning point at the end, and this was attributed to the local flange delamination near the stain gage.

In conclusion, based on the strain readings measured on both the flange and web plates, the proposed curved-pultruded GFRP arch beams are exempt from the flange- and web-related buckling issues, confirming the test result that these beams achieved their strength limit states.

5. Analytical program

Conventionally pultruded GFRP beams may sustain serious local and global buckling issues, including FLB, WLB, web crippling, web compression buckling, LTB and LSD affected-LTB. The buckling-induced failures could significantly reduce the load-carrying capacities of GFRP beams. In addition to the experimental tests, an analytical study was conducted to investigate the possible buckling behaviors. In particular, the LTB and LSD affected-LTB were not discussed herein, and a good correlation between experimental and analytical results was observed.

In light of the study by Borowicz and Bank [17], Kollár’s equation for fixed web plate [61] was adopted in this work, as shown in Eq. (1).

\[ f_{cr,WLB} = \frac{d^2}{(d-t_f)} t_w \left[ 26.8 \sqrt{D_w} + \frac{12.9(D_w^2 + 2D_{16})}{D_{12}^2} \right] \]  

where \( f_{cr,WLB} \) is the critical web local buckling strength; \( d \) is the section depth; \( t_f \) is the flange thickness; \( t_w \) is the web thickness; \( D_{ij} (i, j = 1, 2, 6) \) is the flexural stiffness parameter of the homogenous orthotropic plate (see Appendix A for calculations of \( D_{ij} \)), and the superscript \( w \) indicates the web plate.

The critical web crippling and web compression buckling (WCB) loads \( P_{cr} \) were calculated in accordance with ASCE [1], as shown in Eqs. (2) and (3), respectively. In particular, to simplify the calculation as well as to achieve a conservative prediction, the term in brackets of Eq. (2) was neglected in the calculation of the critical web crippling load; that is, the contribution of the bearing plate was neglected.

\[ P_{cr,crimping} = 0.7d t_w F_{th,lim} \left( 1 + \frac{2k + 6t_{plate} + t_{plate}}{d - t_f} \right) \]  

\[ P_{cr,WCB} = \frac{\pi^2 E_w t_w^2}{6(d-t_f)} \left[ \sqrt{E_w^2 E_f^2 + E_f^2 n_{LT} + 2n_{LT}^2} \right] \]
Fig. 8. Failure modes of curved-pultruded GFRP arch beams.

a) SS-7

b) SS-12A

c) SS-12B

Fig. 9. Flange and web strain distributions of SS-12B.
reaching the ultimate loads (evidenced by the top plates were observed not to sustain any noticeable rotations prior to greater than most of the market relatively high rotation 2.1, the FLB strength of pultruded GFRP I cates the web plate. Top strains at mid-span of curved-pultruded GFRP arch beams. In addition, the interior and exterior FLB strengths, denoted as fcr,intFLB and fcr,extFLB, respectively; bf,int is the width of the interior flange plate, and in this case, bf,int is 185 mm (= 200 – 15 mm); Df (i, j = 1, 2 and 6) is the flexural stiffness parameter of the homogenous orthotropic plate (see Appendix A for calculations of Df), and the superscript f indicates the flange plate; bext is the width of the exterior outstanding flange plate, and in this case, bext is 100 mm (= 200/2 mm); and factors K and V are calculated as follows [37]:

\[
K = (D_{ij} + 2D_{66}) / \sqrt{D_{11}D_{22}}
\]

\[
V = D_{ij} / (D_{11} + 2D_{66})
\]

With buckling-induced failure modes established, the material strength-related failure mode was also analyzed. Based on the experimental observations, all three beams, SS-7, SS-12A and SS-12B, failed due to compression flange delamination (CFD). Thus, the measured compressive strength of material f_{EL} (see Table 1) was used to calculate the flexural strength of the beam, as shown in Eq. (8).

\[
M_{cr, CFD} = f_{EL}S
\]

where S is the section modulus of the beam.

Fig. 10. Web strains at mid-span of curved-pultruded GFRP arch beams.

where \( P_{cr, crippling} \) and \( P_{cr, WCB} \) are the critical buckling loads due to web crippling and web compression buckling, respectively; \( F_{el, int} \) is the interlaminar shear strength; \( k \) is the distance from the top of the top flange to the bottom of the fillet and is calculated as \( (t_f + t) \); \( t_{plate} \) is the thickness of the bearing plate; \( b_{plate} \) is the length of the bearing plate; and \( E_L, E_T \) and \( G_{LT} \) are the longitudinal, transverse and in-plane shear modulus of the material, respectively, and the superscript w indicates the web plate.

In addition, the interior and exterior FLB strengths, denoted as \( f_{cr, intFLB} \) and \( f_{cr, extFLB} \), respectively, were calculated. As discussed in Section 2.1, the FLB strength of pultruded GFRP I-beams is significantly affected by the restraining effect of flange-web junctions [9,10,43]. Typically, an elastic flange-web junction is assumed, indicating that the flange plate is permitted to rotate with respect to the flange-web junction. In this work, however, both the interior and exterior flange plates were observed not to sustain any noticeable rotations prior to reaching the ultimate loads (evidenced by the top flange strains shown in Fig. 11). Indeed, the web plates were specifically designed to have a relatively high flexural stiffness; that is, the web thickness of 15 mm is greater than most of the market-available GFRP profiles. Since the rotational restraint to the flange plate is primarily provided by the web plate through its flexural stiffness, it can be rationally assumed that the greater-than-usual thickness of the web is able to provide a substantially improved rotational restraint to the flange plate. With that said, similar to the web plate, a fixed boundary condition was assumed for the flange plates instead of the elastically rotationally restrained boundary condition. Again, Kollár’s equations [37] for doubly- and singly-fixed plates were adopted for interior and exterior flanges, as shown in Eqs. (4) and (5), respectively.

\[
f_{cr, intFLB} = \frac{k^2}{b_{int}^2} \left[ \frac{4.53}{D_{11}}D_{22}^{1/2} + 2.44 \left( \frac{D_{12} + 2D_{66}}{2} \right) \right]
\]

\[
f_{cr, extFLB} = \frac{\sqrt{D_{11}}D_{22}^{1/2}}{b_{ext}^2} \left[ 15.1K \frac{1}{1 - V^2} + 7 \right]
\]

5.2. Results and discussion

Converting all types of flexural strength into moment capacities \( M_{cr} \) via equations \( M_{cr} = f_{el}S \) or \( M_{cr} = F_{el}/4 \), a comprehensive comparison between the experimentally and analytically determined flexural strength of curved-pultruded GFRP arch beams can be made, as shown.
in Table 3 and Fig. 12. Through comparison, it is seen that the buckling-dominated flexural strength, including WLB, web crippling, web compression buckling and interior and exterior FLB, are all greater than the material strength-dominated flexural strength, the compression flange delamination. This correlates well with experimental observations that all beams failed due to the material damage of the compression flange. Thus, in the analytical study, the effectiveness of the proposed curved-pultruded GFRP arch beam is demonstrated.

6. Numerical program

6.1. Finite element modeling

A finite element (FE) modeling was conducted via ABAQUS [60] to investigate the flexural behavior of the proposed curved-pultruded GFRP arch beam as well as to validate the experimental and analytical results. In the above analytical study, the curvature of the beam was neglected and that assumption was validated in this section. In construction of FE model, the three GFRP I-sections of each beam, assembled together using steel bolts and epoxy adhesives in the tests, were set as an integral section. Shell element S4 was used to model the GFRP I-beams. Orthotropic material properties obtained in the material characterization tests (see Table 2) were adopted. Hashin damage criteria were used to define the strength limit state of the GFRP material. The FE model exactly mirrored the actual experimental test set-ups; that is, a simply-supported boundary condition was defined, and a concentrated load was applied at the top of the mid-span via displacement control. In addition, a characteristic element size was first determined through a mesh convergence test to achieve the most efficient computational cost as well as yielding a satisfactory accuracy. A total of two curved beams, SS-7 and SS-12, were modeled. The meshing of SS-7 and SS-12 is shown in Fig. 13. Moreover, two straight beams with spans of 7 and 12 m, denoted as SS-7 straight and SS-12 straight, respectively, were also modeled to investigate the possible influence of the curvature on the flexural behavior of the beam.

6.2. Results and discussion

Through FE modeling, the ultimate failure modes and the load-carrying capacities of the curved-pultruded GFRP arch beams were obtained, as shown in Fig. 13. Again, the longitudinal compressive damage on the top flange marked the ultimate failure of the beam. The FEM results were further compared with the experimental results, and the curvature of the beam was studied, as shown in Fig. 14. First, for the arch beams, an excellent agreement between the experimental and numerical results is seen: 1) the experimentally and numerically determined moment capacities are 1509 and 1556 kNm for SS-7, respectively, showing a difference of approximately 3%; 2) those two moment capacities are 1759 and 1630 kNm for SS-12A, respectively, showing a difference less than 8%; and 3) those two moment capacities are 1662 and 1630 kNm for SS-12B, respectively, showing a difference less than 2%. Through this comparison, the experimentally observed failure modes can be validated. As for the curvature of the beam, Fig. 14 shows that the flexural behaviors (strength and stiffness) between curved and straight beams (SS-7 and SS-12) are negligible, thus demonstrating the validity of analyzing the curved beams using the methods of straight beams.

7. Prospects of Curved-Pultruded GFRP arch beams

In addition to the experimental study presented in the above sections, a full-scale pedestrian bridge over a span of 20 m was designed and constructed using the proposed curved-pultruded GFRP arch beam, as shown in Fig. 15. In this bridge, a total of 11 I-sections were

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Experimentally determined flexural strength (kNm)</th>
<th>Analytically determined flexural strength (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Web local buckling (Eq. (1))</td>
<td>Web crippling (Eq. (2))</td>
</tr>
<tr>
<td>SS-7</td>
<td>1509</td>
<td>2584</td>
</tr>
<tr>
<td>SS-12A</td>
<td>1759</td>
<td>2209</td>
</tr>
<tr>
<td>SS-12B</td>
<td>1662</td>
<td>2584</td>
</tr>
</tbody>
</table>

Fig. 12. Comparison of experimentally and analytically determined flexural strength.
used. The total width of the flange was 2200 mm, and the height was 600 mm. The identical beam assemblage scheme to SS-7, SS-12A and SS-12B was adopted, as shown in Fig. 5b. The beam ends were fixed by a pair of tailor-designed steel end-supports. In addition, the steel end-supports were connected through two steel tension members. Thus, in fact, this bridge took the form of a beam-string structure. An axial compression force was therefore provided to the beam when the bridge was loaded, making it an arch-like bridge.

This bridge was tested under the service limit state. Two Chinese design codes for municipal and highway bridges, CJJ69 [21] and JTG D60 [34], respectively, have prescribed the design pedestrian loads: 1) for municipal pedestrian bridges, the design load is 4.725 kN/m², resulting in a total load of 204 kN; and for highway pedestrian bridges, the design load is 3.5 kN/m², resulting in a total load of 151 kN. In addition, according to the Chinese design guide CECS [20] the maximum mid-span deflection of the pultruded FRP beam with a span over 5 m is L/250 (80 mm, in this case). In this work, the load was conservatively taken as 7 kN/m², and thus, a total load of 303 kN was applied on the bridge, which was realized through the lift trucks shown in Fig. 15a. At the design load, no damage was observed in the GFRP beams, and the vertical deflection at mid-span was measured as 49 mm, which was less than the required maximum 80 mm, thus satisfying the deflection criterion. In addition, according to CJJ69 [21], the minimum natural frequency of vibration for municipal pedestrian bridges is 3 Hz. In this test, the proposed bridge showed a vibration frequency of 20 Hz when the bridge was fully loaded, greater than the required minimum value. Thus, this GFRP pedestrian bridge is able to satisfy the service requirement and provide an excellent load-carrying capacity.

This 20-m-long pedestrian bridge, installed as a corridor between two buildings as show in Fig. 15b, successfully showed the great prospects of the proposed curved-pultruded GFRP arch beam. Through the
curved-pultrusion technique, more advanced structural forms can be anticipated for GFRP profiles, such as the shallow arch and beam-string structure. With the advantages of high strength- and stiffness-to-weight ratios, together with the possibility of advanced structural forms, curved-pultruded GFRP profiles are believed to have wider applications in the field of civil engineering.

8. Conclusions

In this work, an innovative curved-pultruded GFRP arch beam was proposed to improve the flexural performance of FRP structures. The excessive deflections and premature buckling failures were identified as the most critical factors limiting the applications of FRP structures. To overcome these two issues, the latest curved-pultrusion technique was adopted, and a new type of GFRP arch beam was developed. In the end, a full-scale GFRP pedestrian arch bridge was designed, constructed and tested, demonstrating the great potential of the proposed curved-pultruded GFRP arch beam. In this work, the following conclusions can be drawn.

1) The curved-pultrusion technique enables an arch/cambered GFRP beam, thus permitting an offset to the excessive deflection of the GFRP beam. Compared to the conventionally pultruded GFRP beams, namely straight beams, a curvature is introduced to the beam in curved-pultrusion process, through which the clearance under the structure can be ensured and the service limit state can be satisfied.

2) The curved-pultrusion technique enables the production of large-scale GFRP beams with large sections. In this work, an I-beam with a height of 600 mm was successfully manufactured, permitting a high flexural stiffness and a longer span length. In addition, with the beam assemblage scheme proposed in this work, the span length of GFRP structures can be further increased.

3) The designed flange and web slenderness ratios could successfully avoid the possible flange- and web-related buckling behaviors. The web plate was specifically thickened to have a high slenderness ratio and to provide a greater restraint to the flange plate. Additionally, three I-sections were combined together to eliminate the free edges of the flange plate and achieve a high lateral stiffness of the beam. In the experimental tests, the proposed I-beams were observed to be exempt from flange- and web-related buckling failures, and the material strength-dominated failure mode, namely compression flange delamination, was achieved.

4) The large radius of curvature of the proposed GFRP arch beam permits the use of the design equations for straight beams. The radius of curvature was designed to be 56,006 mm, thus yielding an arch rise of 322 mm for the 12-m-long beam. Through both analytical and numerical studies, the validity of using the design equations of straight beams was proven.

5) The prospects of the proposed curved-pultruded GFRP I-beam are successfully demonstrated through a 20-m-long pedestrian bridge. Using the proposed beam, a full-scale pedestrian bridge was designed, constructed and tested. This bridge exhibited excellent performance, satisfying all the strength, deflection and vibration requirements. This bridge took the forms of a beam-string structure and a shallow arch, enabling an improved stress flow in the beam. Thus, this bridge shows a great potential for incorporating advanced structural forms into GFRP structures.

9. Data availability

All data, models, and code generated or used during the study appear in the submitted article.

CRediT authorship contribution statement

TianQiao Liu: Methodology, Investigation, Data curation, Formal analysis, Visualization, Writing - original draft, Writing - review & editing. Peng Feng: Conceptualization, Methodology, Supervision, Project administration, Funding acquisition, Validation, Writing - review & editing. Yuwei Wu: Investigation, Software, Validation, Visualization. Shuxin Liao: Investigation, Data curation. Xinmiao Meng: Investigation, Data curation, Validation.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Acknowledgments

This work was supported by grants from the China National Key Research and Development Project (No. 2017YFC0703000) and the National Natural Science Foundation of China (No. 51908321 and No. 51522807). The authors would give special thanks to Beijing...
Appendix A

The flexural stiffness parameters of the orthotropic plate are calculated as follows [12]:

\[
D_{11} = \frac{E_c t^3}{12(1-\nu_{12}\nu_{21})} \\
D_{22} = \frac{E_c t^3}{12(1-\nu_{12}\nu_{21})} \\
D_{12} = \nu_{21}D_{22} \\
D_{66} = \frac{G_{c}t^3}{12}
\]

References


