Strength and Stiffness of Adhesively Bonded GFRP Beam-Column Moment Resisting Connections

Ascione F[1,*], Lamberti M[2], Razaqpur AG[3], Spadea S[4]

[1] Dept. of Civil Engineering, University of Salerno Italy, email: *fascione@unisa.it*
[2] Dept. of Civil Engineering, University of Salerno Italy, email*: malamberti@unisa.it*
[3] Dept. of Civil Engineering, McMaster University, Hamilton, Canada, email: *razaqpu@mcmaster.ca*
[4] Dept. of Architecture and Civil Engineering, University of Bath, United Kingdom, email: *s.spadea@bath.ac.uk*

**Corresponding author*

Abstract
For the first time, the feasibility of adhesively bonded connections in FRP frame structures is explored as an alternative to bolted connections. Eight full-scale GFRP beam-column connections are tested and their failure mode, strength and rotational stiffness are investigated. A single pultruded GFRP I-profile is used for the two members. In four of the specimens the beam and the column are connected by epoxy adhesive and GFRP seat angles, similar to the so-called "standard bolted connection". In the remaining four specimens, the seat angles are supplemented by additional GFRP angles and stiffeners to strengthen the column flange and web. The beam-column assembly forms an inverted L-shape frame, with the column being fixed at the bottom and attached to the beam near the top. The beam, acting as a cantilever, is loaded by a point load near its free end, which subjects the connection to bending and shear. The current standard connection failed by debonding within the column flange while the improved/strengthened connection failed within the adhesive or at the adhesive-column flange interface. The test results reveal that both the standard and improved connection can have at least the same strength as the corresponding bolted connection, irrespective of whether GFRP or steel bolts are used to make the connection. Hence, the current restrictions against the use of adhesive beam-column connections in GFRP frame structures may be unjustified. In making this comparison, the observed failure load of each connection is normalized by the ultimate moment capacity of the GFRP profile in the beam-column assembly.

Keywords: Adhesive, Connections, Failure moment, GFRP, Seat angle, I-profile, Rotation, Stiffness.

1. Introduction

The use and applications of composite structures made of FRP (Fibre Reinforced Polymers or Plastics) are increasing due to the FRP favorable properties. Among these, one can point out rapid on-site assembly, lightweight, high resistance to aggressive chemicals, superior fatigue life and electromagnetic neutrality. The FRP lightweight offers particular advantage in the case of structures resting on weak soils while their high strength to weight ratio allows for greater load bearing compared to structures made of conventional building materials.
FRP products are available in the form of reinforcing bars or prestressing cables for concrete structures, as sheets, laminates, gratings, grids and structural profiles. The use of FRP profiles is appropriate and advantageous for applications in construction, especially for the construction of

industrial and low-rise residential buildings as well as temporary structures built in emergency situations. The latter could be a source of future expansion for the application of full composite structures.

For FRP composite structures to be competitive with structures made of traditional materials, they must be safe, serviceable, durable and economical. Structural safety and serviceability depend on the structural members', as well as on their joints' or connections', strength and stiffness. The connections must be able to safely resist reasonable levels of load compared to the members' load-bearing capacity, and must possess sufficient stiffness to avoid excessive deformations.

Currently, the connections in FRP structures are commonly made using bolted connections, akin to those used in steel structures. In fact, some FRP profiles manufacturers and design guidelines either specifically prohibit the use of adhesive/bonded connections or restrict their application. For example, the FiberLine Design Manual **Errore. L'origine riferimento non è stata trovata.** discourages the use of bonded structural connections ostensibly due to lack of experience and clear design rules for such connections in complex cases. Also, a recent European report **Errore. L'origine riferimento non è stata trovata.** by Technical Committee 250 of CEN (Comité Européen de Normalization) stipulates that bonded connections should not be allowed for primary load bearing components, where failure of the connection could lead to progressive collapse or unacceptable risks. In the latter situations, their use is permitted only in combination with or as a backup for bolted connections. At the same time, the report highlights that prohibition of bonded connections represents one of the key issues that impede the steadily increasing market for FRP profiles in the field of civil construction, which already utilizes 35% of the annual world-wide production of GFRP profiles.

As stated in **Errore. L'origine riferimento non è stata trovata.**, the main reason for the prohibition of bonded connections is lack of knowledge about and experience with the performance of such connections. Hence, there is need for research on bonded connections in order to understand their behavior, in terms of strength and stiffness, and to assess their performance vis-à-vis similar bolted connections. The knowledge thus gained can be used by designers to safely design composite structures with bonded connections, provided that it can be demonstrated to be more advantageous than using bolted connections.

Theoretically, there are reasons to believe that bonded connections can be superior to bolted connections in FRP composite structures. For example, it is well known that the holes made in structural members with bolted connections cause stress concentration and increase the risk of moisture penetration in members. Also, as discussed later in this paper, available research has shown that current moment resisting bolted beam-column connections in GFRP structures can rarely resist more than 20% of the flexural capacity of the connected members **Errore. L'origine riferimento non è stata trovata.**,[4],[7]. On the contrary, it has been observed in simple bonded lap joint connections that due to the absence of holes, the stresses are more uniformly distributed over the bonded surfaces, stress concentration and damage to the fibers caused by the holes are non-existent. While bonded lap joints have been extensively investigated in composites, for beam-to-column bonded connections there are no experimental or numerical results available to assess their strength, stiffness and overall performance. To fill this gap in knowledge, the objective of this paper is to experimentally investigate the behavior of full-scale bonded beam-to-column moment resisting connections, with the ultimate goal of developing high performance bonded connections that can equal or surpass the performance of similar bolted connections in FRP structures. Based on the authors' knowledge, this is the first study of its kind and therefore its scope is limited to monotonically increasing static loading of the connection, without dealing with issues of creep or fatigue, which would be subjects of future investigations.

2. Literature review

To put the current study in its proper context, a brief review of previous research on beam-column connections in FRP structures is presented. In the early nineties, Bank et al. **Errore. L'origine**

riferimento non è stata trovata.,[4] tested pultruded GFRP I-beam and column, connected by GFRP bolts, pultruded seat angles and web clips, as shown in Fig. 1(a), or they strengthened the facing flange of the column in the joint region by GFRP angles, Fig. 1(b), and then used the same type of connecting elements as in (a). Finally, to mitigate the failure mode initiated by the separation of the column web from the facing flange in the region of the top seat angle, they stiffened the beam and column in the connection region, by adhesively bonded GFRP tubular sections, and joined the members by FRP gusset plates and bolts as in Fig.1(c). They concluded that the design of beam-column connections in pultruded FRP members requires careful consideration of the unique local failure modes occurring either at the web-flange junction or in the web transverse to the direction of pultrusion.

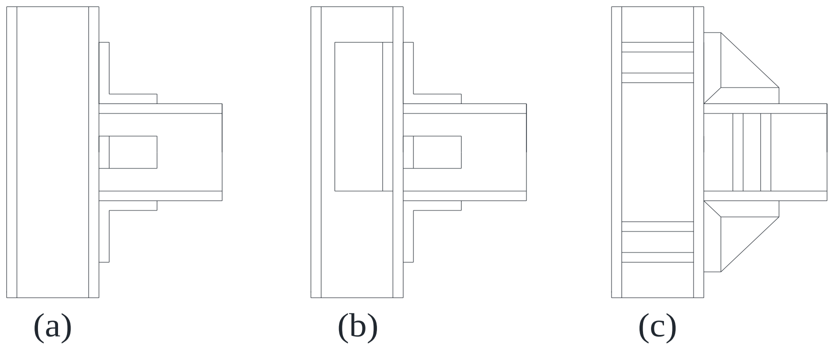


Figure 1. Connections [3,4]: a) standard joint, b) reinforced column flange and c) gusset plate angles.

In 1994, Mosallam [5] and Mosallam et al.[6] pointed out that it is not appropriate to design FRP frame connections using concepts developed for connections in steel structures. Instead, they developed a built-up GFRP connecting element termed “universal connector”, Fig. 2, bolted to the beam and column. The typical failure mode in the latter connection was a combination of local failure in the universal connector and punching of the bolts through the column flanges. This connection increased joint strength almost threefold over the standard seat angle joint.

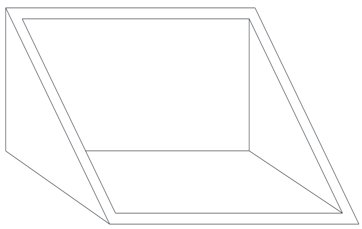


Figure 2. Universal connector [5,6].

Bank et al. **Errore. L'origine riferimento non è stata trovata.** modified the seat angles by wrapping them with a GFRP sheet, Fig. 3, and used this so-called wrapped connector to bolt the beam to the column. They indicated that, compared to other types of connections previously tested **Errore. L'origine riferimento non è stata trovata.**,[4], their proposed connection yielded the best combination of strength, stiffness, failure modes and construction practicality. However, although the stiffness was much higher for this connection compared to the one in [5],[6], it should be pointed out that the failure load was almost 30% lower.

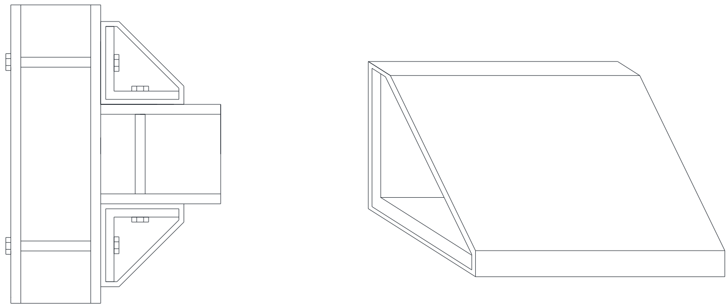


Figure 3. Wrapped connection [7].

In 1998, Smith et al. [8] presented the results of an experimental investigation on pultruded GFRP I and box sections. The tests comprised six specimens: three beams and columns with I-profile and three with hollow or box section. In the case of the I-section, the beam-column connection was almost identical to the one used by Bank et al. **Errore. L'origine riferimento non è stata trovata.**, but instead of GFRP bolts, steel bolts were used. In one case, the GFRP seat angles were replaced by steel angles. The box sections were joined on the top and bottom faces of the beam to the column by GFRP angles and on the sides by GFRP plates. In all cases steel bolts were used to join the elements. With respect to both strength and stiffness, the box beam connections performed much better than the I-beam connections. The results, in terms of frame stiffness and ultimate load, will be presented later in this paper for comparison with those obtained in the current study.
In 1999, Smith et al. [9] presented the findings of an experimental investigation to demonstrate the performance of their proposed monolithic connecting element, termed “cuff” (Fig. 4). The element was used to connect GFRP box beams and columns and led to substantial increases in joint stiffness (90%) and strength (330%), compared to the earlier typical seat angle connections used to join GFRP I-beams and columns. The concept of the cuff connection is that the beam and column can fit into the hollow cuff sections, ideally requiring only epoxy to keep them in place, albeit Smith et al. used both steel bolts and adhesive to join the beam and column to the cuff.

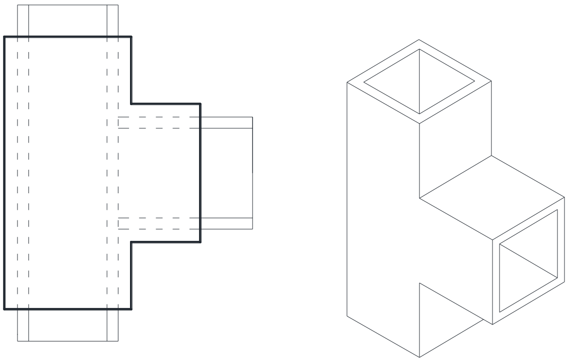


Figure 4. Idealized cuff connection [9].

In 1999, Mottram and Zheng [10],[11] conducted an experimental investigation on an interior connection, involving two cantilever beams connected to a central column. They studied the stiffness and behavior of web-cleated and flange-cleated beam-to-column connections. In the former, only GFRP clips were used in conjunction with steel bolts to join the beams and the column while in the latter they used GFRP seat angles and steel bolts. They concluded that the connection satisfied the requirements of the Eurocomp design code [12],[13].
As a follow up to the latter study, Quareshi and Mottram [14],[15] focused on the number and location of required steel bolts in the connection and concluded that only two bolts are required to achieve satisfactory connection performance. They also investigated the behavior and failure mode of the connection when the cleat was made of either steel or GFRP. They concluded that steel cleats lead to connection failure by the general failure of the column while with GFRP cleats failure is

initiated by local delamination in the column flange above the cleats.

As it may be noticed, all the existing research works involving GFRP beam-column connections have used either steel or GFRP bolts, but no study has been conducted on purely adhesive/bonded joints. However, past studies have investigated the behavior of single lap and double lap adhesive joints between pultruded composite flat profiles. The experimental and numerical works carried out by Sarfaraz et al. [17], Hai et al. [18], Kumar et al. [19], Li et al. [20], Ascione [21],[22] are representatives of these investigations. These studies focus on the influence of the mechanical and geometrical properties of the adhesive and adherents, as well as the exposure conditions, on the rupture and/or fatigue behavior of joints. Although they are important works, due to the significant differences between the type and distribution of stresses in lap joints versus beam-column joints, it is difficult to extrapolate the lap joints results to the kind of moment and shear resisting beam-column connections tested in the current investigation. Therefore, in the present study an experimental program is undertaken to evaluate the behavior, strength and failure modes of bonded beam-column connections in GFRP pultruded I-profiles subjected to combined shear and moment.

3. Experimental Program

Test Specimens and Their Connections

A total of eight full scale specimens were identically tested. A single commercially available pultruded GFRP 200x100x10 mm I-profile **Errore. L'origine riferimento non è stata trovata.** was used to make both the beam and the column in all the specimens. As illustrated in Fig. 5, the column and beam had total length of 1000 mm and 500 mm, respectively. In all cases, the beam was bonded to the column flange, thus both the beam and the column were subjected to bending about their strong axes. In each specimen the beam flanges and web were connected to the compression flange of the column by 50x50x6 mm GFRP seat angles (L-profiles) using epoxy adhesive.

Table 1. Mechanical and geometrical properties of the profiles **Errore. L'origine riferimento non è stata trovata.**

Profile	Geometry	Mechanical properties				
	[mm]	Elastic Modulus,0° [MPa]	Elastic Modulus,90° [MPa]	Shear Modulus [MPa]	Flexural Strength,0° [MPa]	Tensile Strength,90° [MPa]
I	200x100x10	28000	8500	3000	240	50
L	50x50x6	23000	8500	3000	240	50

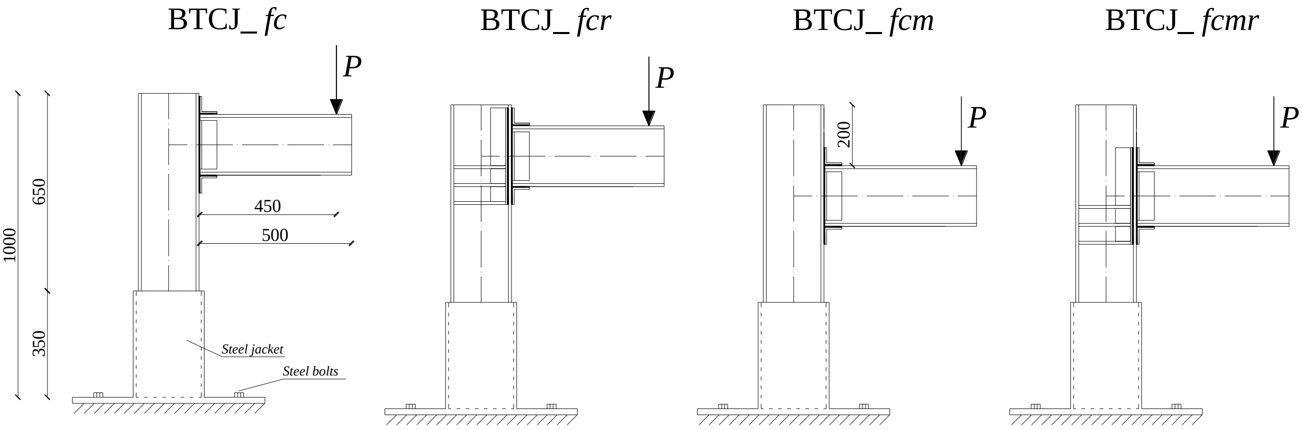
The detailed geometric and mechanical properties of the I- and L- profiles, including their strength and elastic modulus, as specified by the manufacturer, are given in Table 1. A characterization of the I-profile's modulus of elasticity by flexural tests, according to UNI EN 13706-2, can be found in a recent work by Ascione et al.[16]. The pertinent properties of the SikaDur30 epoxy are given in Table 2. Note that the curing time and temperature range for this epoxy are specified by the supplier as seven days and 22-28 ^{o}C, respectively. In the current tests these specifications were complied with.

Table 2. Epoxy adhesive (SikaDur30) properties.

SikaDur30		Value [MPa]
Young Modulus	E_c	9600
	E_t	11200
Compressive strength	σ_c	80-85
Tensile strength	σ_t	26-27
Shear strength	τ	16-17

Four connection types were tested, with the test parameters being the location of the connection with respect to the free end of the column and the column reinforcement. They are designated as BTCJ_*fc*, BTCJ_*fcr*, BTCJ_*fcm* and BTCJ_*fcmr*, where BTCJ stands for Beam-to-Column

Junction, *fc* for flange connection, the letters *m* and *r* for middle and reinforced, respectively. The connections details are shown in Fig. 5 (a) to (d). In the BTCJ_*fcr* and BTCJ_*fcmr* specimens the column was strengthened in the connection region with adhesively bonded pultruded 70x45x10 mm GFRP strips and 50x50x6 mm GFRP angles, as illustrated in Fig. 6, while in specimens BTCJ_*fc* and BTCJ_*fcm* no such reinforcement was used. The strips were obtained by cutting a portion of the L-profile flange, whereas the L-profiles used to strengthen the column were identical to the ones used as seat angles (Table 1).



(a) (b) (c) (d)

Figure 5. Details of the beam-column connections tested: a) BTCJ_*fc*, b) BTCJ_*fcr*, c) BTCJ_*fcm* and d) BTCJ_*fcmr*.

The beam was connected to the column either near the top of the column as in Fig. 5 (a) and (b) or at 200 mm below the top as in Fig. 5 (c) and (d), i.e near the mid-height of the clear length of the column. The connection position was varied with respect to the free end of the column in order to investigate the effect of the proximity of the connection near the free end of the column on the column strength and failure mode. Column reinforcement was used to determine whether prior to the actual connection failure, failure initiated by delamination at the column web-flange connection, as reported in the case of bolted connections **Errore. L'origine riferimento non è stata trovata.**,[4],[5],[6],**Errore. L'origine riferimento non è stata trovata.**,[8],[9], can be averted. Finally, to ascertain the repeatability of the results, in each case duplicate specimens were tested.

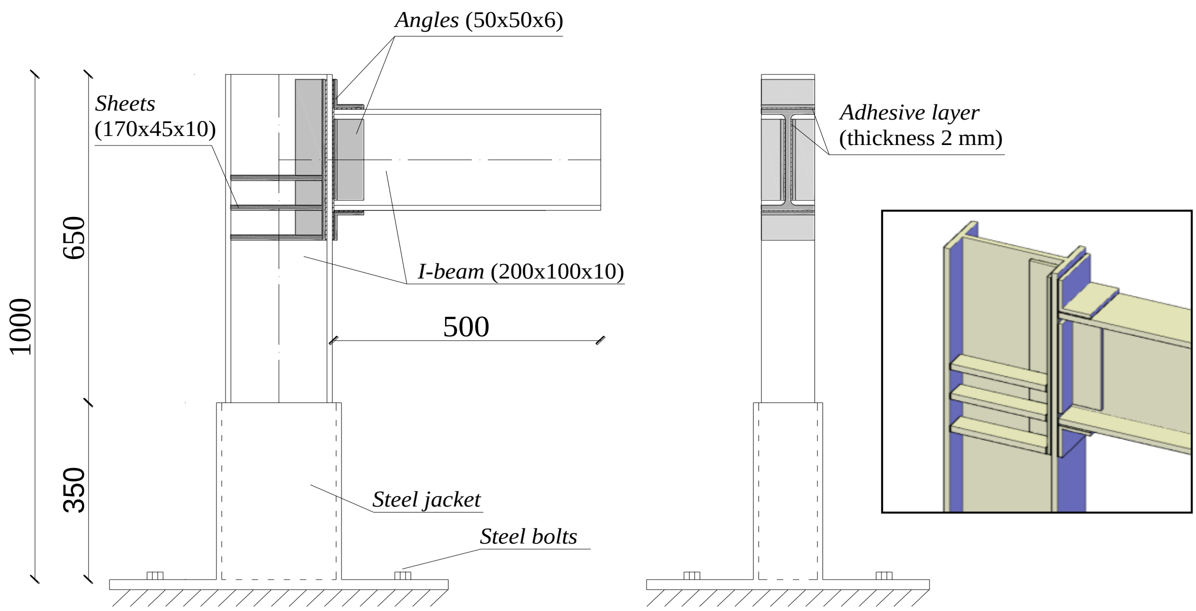


Figure 6. Column strengthening details in the connection region.

Instrumentation and Test Set-up

To evaluate the deformations, stiffness and failure load of the members and their connection, beam and column displacements were monitored at a number of locations. As illustrated in Fig. 7, three LVDTs, designated as LV1, LV2 and LV3, were placed along the length of the beam to measure its vertical displacement while two horizontal LVDTs, denoted as LV4 and LV5, were placed against the bottom and top seat angles, respectively, to measure the rotation of the beam at the connection. The latter rotation comprises the rotation of the column at its connection with the beam, assuming a rigid connection, and the additional rotation due to the connection deformability.

Four electrical resistance strain gauges, designated as SG1 to SG4, were placed on the beam near the connection as shown in Fig. 7. One gauge was placed on each of its flanges 60 mm from the column flange while the remaining two gauges were placed on the web 70 mm above and below the beam axis. To capture the transverse normal strain in the column web, as illustrated in Fig. 7, one strain gauge was installed on each face of the column web and are designated as SG5 and SG6.

To monitor possible vertical slip between the beam and the column at their connection, two none contact laser transducers with resolution of one micron, designated as LT1 and LT2, were used to measure the vertical displacement of the beam and the column adjacent to the connection. Finally, a 600 kN load cell was used to measure the applied load near the free end of the beam. All the instrumentation was connected to a computer controlled data acquisition system.

To support the column at its bottom, it was inserted inside a 350 mm high stiff steel jacket welded to a thick steel plate that was bolted to the testing machine platen, Fig. 7. The column fitted snugly inside the jacket and the small gap between them was filled by steel shims. Hence, for all practical purposes, the column can be considered fixed at the bottom with its unsupported length being 650 mm. With reference to Fig. 7, the beam and column assembly was loaded by a point load applied near the free end of the beam at 450 mm from the beam-column flange connected to the beam. The load was transferred through a 20 mm diameter sphere seated in a circular cavity inside a 100 mm diameter plate which rested on the beam top flange.

The beam was loaded monotonically in displacement control at a rate of 1.0 mm /minute by a servo-controlled universal testing machine. All data were automatically and continuously captured.

Figure 8 shows a photo of the typical test set-up.

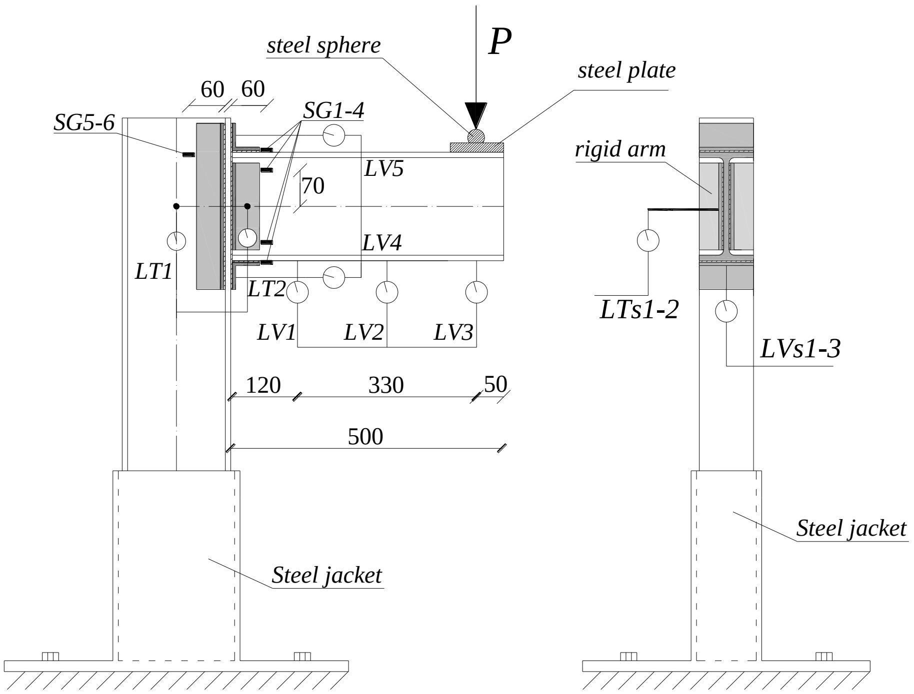


Figure 7. Typical test specimens instrumentation and test set-up.

Figure 8. Typical test specimen's instrumentation and set-up.

4. Experimental results and discussion

In the following, the observed behavior, failure mode, strength and stiffness of each connection type are presented and discussed.

Connection BTCJ_*fc*

This connection failed due to the separation of the column compression flange from the column web in the region of the connection as shown in Fig. 9. The separation spread into the column flange and led to delamination normal to the flange plate. The failure was brittle, but the bond between the beam and the column flange remained intact. This type of failure is similar to that one reported in Feo et al. [23], who tested nominally identical GFRP beams as in the present study to evaluate the longitudinal shear behavior of web-to-flange junctions in an I-profiles subjected to a uniform pull load applied at its bottom flange. The connection exhibited a linear response up to failure as indicated by Fig. 10, which shows the test specimens load-beam tip vertical deflection as recorded by LV3. The plotted values are the average of the values for the two repeat specimens for each connection type as the difference between them was found to be relatively small. The connection reached a maximum applied load of 15.44 kN, which corresponds to an applied moment of 6.95 kN·m at the connection face. The mean failure load and corresponding moment for all the connections are summarized in Table 3. It must be pointed out that the deflection values in Fig. 10 represent the deflection of the beam as a cantilever plus the deflection caused by the column end rotation, the local deformation of the column web and the joint rotation. In reality, the slope of the moment-rotation curve represents the overall rotational stiffness of the beam-column assembly rather than that of the connection alone. Here the connection rotation is computed by taking the relative horizontal displacement measured by gauges LV4 and LV5 divided by the vertical distance (275 mm) between them while local web deformation is evaluated using the data from gauges SG5 and SG6. The latter gauges recorded 2800 $\mu\varepsilon$ average strain at failure. For the given transverse elastic modulus of the GFRP profile, this strain corresponds to 23.8 MPa transverse tension in the web, which is slightly less than half the given tensile strength of the web in the transverse direction. It should be pointed out that transducers LT1 and LT2 indicated essentially zero shear slip at the connection.

The connections moment-rotation curves are plotted in Fig. 11. All the graphs exhibit nonlinearity at low moment levels, followed by a linear part and again ending with a nonlinear segment. The initial nonlinearity is the result of the specimen initial seating and not a characteristic of the connection. Due to the presence of nonlinearity at higher moment levels, a single value for joint stiffness cannot be deduced, however, for comparison we can use the slope of the middle linear segment of the moment-rotation curves. Using the latter slope, for this connection the joint stiffness is computed to be 272 kN·m/rad.

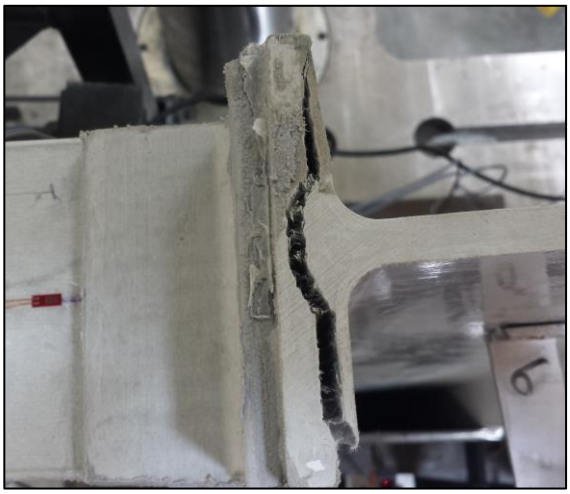


Figure 9. BTCJ_*fc* failure mode.

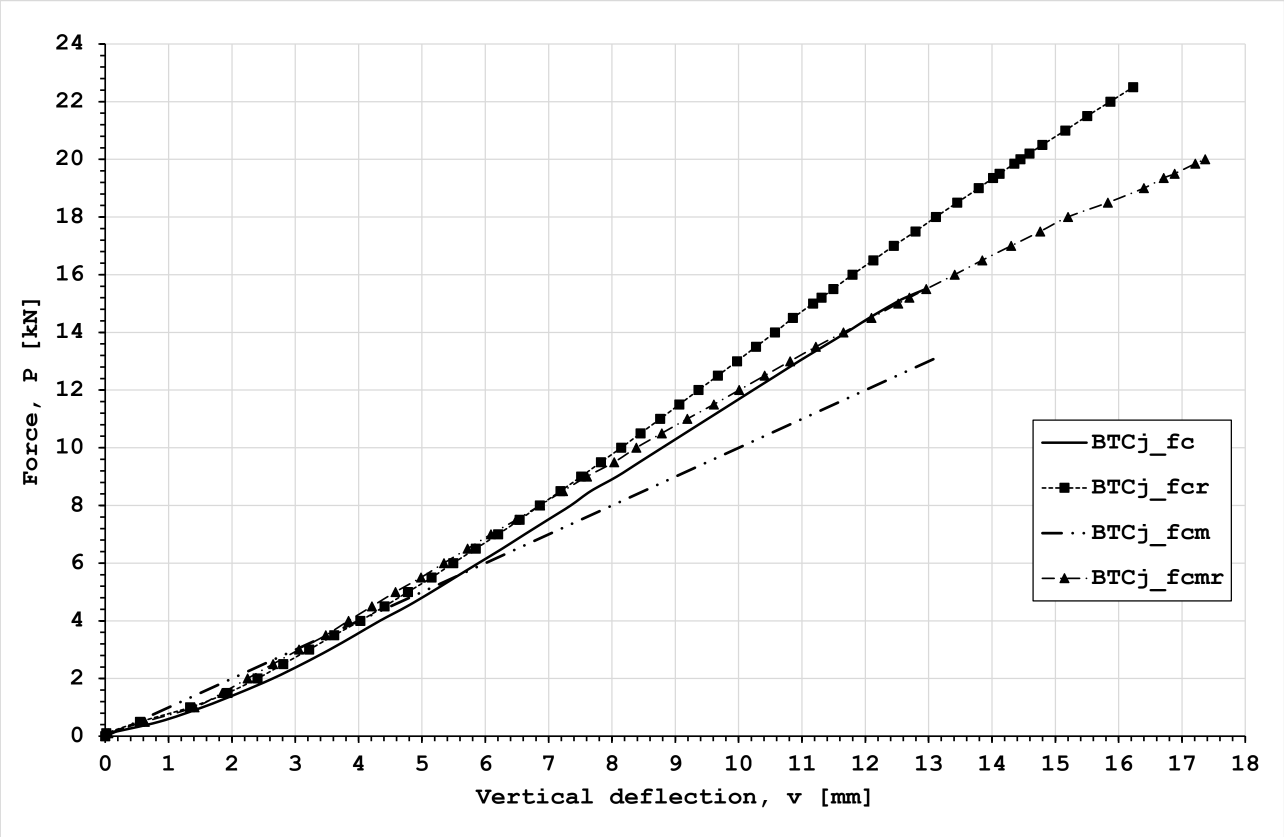


Figure 10. Load-beam tip deflection.

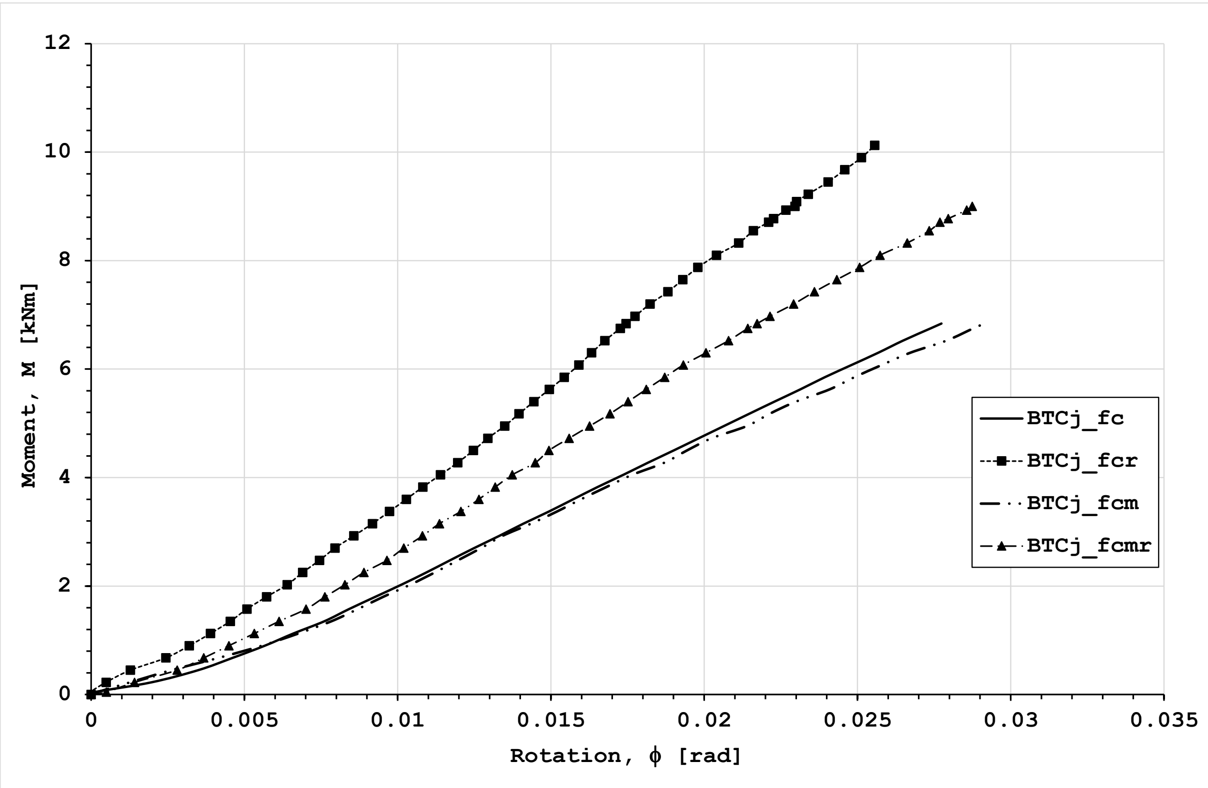


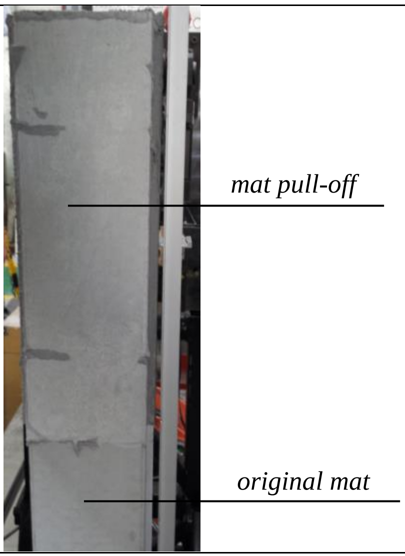
Figure 11. Moment-rotation relationship.

The maximum strain measured by SG1 at failure was 1357 $\mu\varepsilon$, but if we calculate the stress at his point by Navier's formula and assume the elastic modulus of the GFRP profile to be equal to that specified by the profile manufacturer, the corresponding strain would be only 897 $\mu\varepsilon$.

Consequently, in the vicinity of the connection shear deformation occurs in the beam and its behavior deviates from that predicted by the Euler-Bernoulli beam theory. In a subsequent test an additional strain gauge was placed on the tension flange at the beam mid-length to check this phenomenon and it showed that the longitudinal strain/stress closely follow the Euler-Bernoulli beam theory. However, the deviation from this theory within distance *h* from the connection must be considered when designing the connection, where *h* is the beam depth.
Finally, it can be observed that the failure moment is relatively small compared to the bending capacity of the beam or column about its strong axis. Consequently, despite the fact that this connection has significantly higher strength than the standard bolted connection tested by previous researchers, it is still a small fraction of the flexural strength of the connected members. The same observation can be made about the standard bolted connection tested by other investigators. One way of increasing the connection strength is to prevent the column web-flange separation by reinforcing the column in the connection region. The results for such a strengthened column are presented next.

Connection BTCJ_*fcr*

This specimen failed due to the connection failure, which resulted in the separation of the beam from the column. Figure 12 (a) and (b) show after separation the column flange surface in the connection region and the beam bonded end, respectively. The failure plane was at the interface of the column flange and the adhesive and upon closer inspection it appeared to be initiated by the GFRP mat pull-off from inside the column flange. Hence reinforcement of the column changed the failure mode from web-flange separation to mat pullout.
With reference to Fig. 10, the load-deflection relationship is again essentially linear and the average failure load is 22.5 kN, corresponding to a maximum moment of 10.15 kN·m. Based on Fig. 11, the rotational stiffness is approximately 449 kN·m /rad. Therefore, the column reinforcement increased the failure load by 46%, and the rotational stiffness by almost 65%. The maximum strain measured by SG1 at failure was 1579 $\mu\varepsilon$. The average transverse strain in the column web measured by gauges SG5 and SG6 was 1450 $\mu\varepsilon$, which is only 55% of the same strain measured in the companion un-strengthened specimen. Based on these results, when the connection is made near the free end of the column, it is recommended to reinforce the column in the connection region to avoid potential web-flange separation.



(a)	(b)

Figure 12. BTCJ_*fcr* failure mode: a) Column, b) Beam.

Connection BTCJ_*fcm*

In this connection failure was initiated by delamination through the thickness of the column flange, followed by delamination at the interface between the adhesive and the GFRP as can be seen in Fig. 13. With reference to Figs. 10 and 11, the failure load is 15.25 kN, corresponding to a maximum moment of 6.86 kN·m while the rotational stiffness is approximately 249 kN·m/rad. The maximum strain recorded by SG1 is 1084 με. Compared to the similar connection made near the top of the column, the failure load is practically the same, but the stiffness is lower due possible local shear deformations within the column in the connection region.

The average transverse normal strain recorded by SG5 and SG6 was about 2800με, which is practically equal to the corresponding strain in BTCJ_*fc*. The results for this connection, as in the case of BTCJ_*fc*, reveal that an unreinforced column in the connection region is vulnerable to column failure prior to the connection failure; consequently, column reinforcement may be necessary, irrespective of the connection location.

Connection BTCJ_*fcmr*

The failure of this connection was initiated by fracturing of the adhesive layer as shown in Fig. 12. It reached a maximum load of 20.0 kN, corresponding to a maximum moment of 9.0 kN·m at the connection and maximum tensile strain of 1507με in the beam top flange. The rotational stiffness, based on its moment rotation curve in Fig. 11, is 363 kN·m/rad. Compared to the companion connection without column reinforcement, the failure load and rotational stiffness are 31% and 36% higher, respectively. As for the transverse normal strain in the column web, it reached 1551με at failure, which is only half of the same strain in the companion specimen without column reinforcement. The maximum tensile strain recorded by gauge SG1on the beam flange is 1542με. Since in this case the failure was clearly in the connection, it can be argued that this is the limiting strength of this type of connection, unless additional improvements are made. Although the observed failure moment is again relatively small compared to the moment capacity of the beam and the column, still, as shown below, the strength and stiffness values of the reinforced column are either much higher or comparable to those of similar bolted connections tested by previous investigators.

Figure 13. BTCJ_*fcm* failure mode.

5. Comparison of GFRP Beam-Column Bolted and Adhesive Connections

Since previous researchers used GFRP beam and column sections that were not identical to the ones used in the current study, a logical comparison can be made if the reported strengths are presented in dimensionless form by normalizing the failure moment by the moment capacity of the profile

used in each test.
Table 3 shows the aforementioned normalized moment values for the present connections and for the bolted connections tested by Bank et al. [4] and Smith et al. [8],[9]. Each profile moment capacity was evaluated using the flexure or Navier formula together with the geometrical and mechanical properties of each profile as reported by the original authors. The profiles tested by Bank et al. and Smith et al. was produced by Creative Pultrusion and Strongwell, respectively.

Table 3. Connection Test Results

Connection	**Type**	**Investigators**	**Failure load, P_F** [kN]	**Failure moment, M_F** [kN m]	**Profile Theoretical Moment Capacity, M_{th}** [kN m]	$\frac{M_F}{M_{th}}$ [%]
GFRP Bolted	Standard	Bank et al.	-	6.21	106.09	5.85
	Improved	Bank et al.	-	18.41		17.35
Steel Bolted	Standard	Smith et al.	2.9	1.30	53.02	2.45
	Improved	Smith et al.	3.3	1.40		2.64
Adhesive	Standard (BTCJ_*fc*)	present study	15.4	6.95	55.80	12.46
	Standard (BTCJ_*fcm*)	present study	15.2	6.86		12.29
	Improved (BTCJ_*fcr*)	present study	22.5	10.15		18.19
	Improved (BTCJ_*fcmr*)	present study	20.0	9.0		16.13

Note that only the results for the standard and the best performing connections tested by previous researchers are reported. It can be seen that the present standard connections as well as the improved ones, which involved column reinforcement, are comparable with the corresponding standard and improved connections using either steel or GFRP bolts. Surprisingly, the steel bolted connections tested by Smith et al. seem to have very low strength compared to the present adhesive connections or to the steel bolted connections tested by Bank et al[6]. It is evident that adhesive connections can perform as well as or even better than the bolted connections. Unless it is shown otherwise, based on the current results, the complete prohibition by some organizations against the use of adhesive beam-column connections in GFRP frame structures appears to be unjustified.

6. Conclusion

Eight full-scale GFRP adhesively bonded beam-column connections were tested under combined shear and bending and the results of the investigation support the following conclusions:

1) Both the standard and improved adhesive connections can have strength comparable to the corresponding bolted connections, irrespective of whether steel or GFRP bolts are used.
2) The improved connection in the current study, which involved seat angles and strengthened column in the connection region, achieved nearly the same percentage of the GFRP profile ultimate moment capacity as in the best performing bolted connections tested by others.
3) The standard connection in the current tests failed, in one case, due to debonding in the column flange, a failure mode similar to that reported for the standard bolted connection. On the other hand, the improved connections failed due to either cohesive failure within the adhesive or the adhesive-column flange interface.
4) Column reinforcement can increase the connection failure load by approximately 40%.
5) To avoid premature failure at the connection initiated by column web-flange separation, the column needs to be reinforced in the connection region, as demonstrated in this

investigation.

6) The location of the beam connection to the column, relative to the free end of the column, does seem to have a significant effect on the connection strength.

Acknowledgments

The authors thank the Italian Department of Civil Protection (DPC), which provided financial support to this research (Project ReLuis 2016) and the Natural Sciences and Engineering Research Council of Canada for its support through its Discovery Grant RGPIN-2016-05897. They would also like to thank Messrs Alessandro Guariniello, Salvatore Oliva and Fabrizio Zuccaro, undergraduate students at the Department of Civil Engineering at the University of Salerno, for their assistance during the preparation and testing of the specimens.

References

[1] Fiberline Design Manual. https://fiberline.com/sites/default/files/mediafiles/190107_dm_uk.pdf

[2] Report EUR 27666 EN. Prospect for new guidance in the design of FRP. JRC Science for Policy Report, 2016.

[3] Bank LC, Mosallam AS, Gonsoir HE. Beam-to-column connections for pultruded FRP structures. Suprenant B editor. Serviceability and durability of construction materials, Proceedings of the First Materials Engineering Congress, Denver, CO, 13-15 August, ASCE, 1990:804-813.

[4] Bank LC, Mosallam AS, McCoy GT. Design and performance of connections for pultruded frame structures. Journal of Reinforced Plastics and Composites 1994;15:1052-1067.

[5] Mosallam AS. Stiffness and strength characteristics of PFRP UC/beam-to-column connections. Composite material technology, PD-vol.53, Proceedings, ASME Energy-Sources Technology Conference and Expo, TX, 331 January-4 February, 1993: 275-283.

[6] Mosallam AS, Abdelhamid MK, Conway JH. Performance of pultruded FRP connections under static and dynamic loads. Journal of Reinforced Plastics and Composites 1994;13:386-407.

[7] Bank LC, Yin J, Moore L. Experimental and numerical evaluation of beam-to-column connections for pultruded structures. Journal of Reinforced Plastics and Composites 1996;15:1052-1067.

[8] Smith SJ, Parsons ID, Hjelmstad KD. (1998) An experimental study of the behaviour of connections for pultruded GFRP I-beams and rectangular tubes. Journal of Composite for Construction; 42: 281-290.

[9] Smith SJ, Parsons ID, Hjelmstad KD. (1999) Experimental comparisons of connections for GFRP pultruded frames. Journal of Composite for Construction; 3: 20-26.

[10] Mottram JT, Zheng Y. (1999) Further tests on beam-to-column connections for pultruded frames: web-cleated. Journal of Composite for Construction; 3: 3-11.
[11] Mottram JT, Zheng Y. (1999) Further tests of beam-to-column connections for pultruded frames: flange-cleated. Journal of Composite for Construction; 3: 108-116.
[12] Qureshi J, Mottram JT. (2013) Behaviour of pultruded beam-to-column joints using steel web cleats. Thin-Walled Structures; 73: 48–56.
[13] Clarke, J. L. *Structural Design of Polymer Composites: EUROCOMP Design Code and Handbook*. London: E & FN Spon, 1996.
[14] Qureshi J, Mottram JT. (2014) Response of Beam-To-Column Web Cleated Joints For FRP Pultruded Members. Journal of Composite for Construction; 18.
[15] Qureshi J, Mottram JT. (2015) Moment-rotation response of nominally pinned beam-to-column joints for frames of pultruded fibre reinforced polymer. Construction and Building Materials; 77: 396–403.
[16] Ascione F, Mancusi G, Spadea S, Lamberti M, Lebon F, and Maurel-Pantel A. (2015) On the flexural behaviour of GFRP beams obtained by bonding simple panels: An experimental investigation. Composite Structures; 131: 55-65.
[17] Sarfaraz R, Vassilopoulos AP, Keller T. (2012) Experimental investigation and modeling of mean load effect on fatigue behavior of adhesively-bonded pultruded GFRP joints. International Journal of Fatigue; 44: 245–252.
[18] Hai ND, Mutsuyoshi H. (2012) Structural behavior of double-lap joints of steel splice plates bolted/bonded to pultruded hybrid CFRP/GFRP laminates. Construction and Building Materials; 30: 347–359.
[19] Kumar P, Singh RK, Kumar R. (2007) Joining similar and dissimilar materials with GFRP. International Journal of Adhesion & Adhesives; 27: 68–76.
[20] Li HCH, Dharmawan F, Herszberg I, John S. (2006) Fracture behaviour of composite maritime T-joints. Composite Structures; 75: 339–350.
[21] Ascione F. (2009) Mechanical behaviour of FRP adhesive joints: A theoretical model. Composites Part B: Eng; 40: 116-124
[22] Ascione F. (2010) Adhesive lap-joints: A micro-scale numerical investigation. Mechanics Research Communications; 37:169–172.
[23] Feo L, Mosallam AS, Penna Rosa. (2013) Mechanical behavior of web-flange junctions of thin-walled pultruded I-profiles: An experimental and numerical evaluation. Composites Part B: Eng; 48: 18-39.