Experimental study of GFRP-concrete hybrid beams with low degree of shear connection

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Abstract

Recent developments in the design of advanced composite materials for construction have led researchers to create novel high-performance structural elements that combine fiber-reinforced polymer (FRP) shapes with traditional materials. The current study analyzes the experimental structural response of eight hybrid beams made of pultruded glass FRP (GFRP) profiles mechanically connected to reinforced concrete (RC) slabs, suitable for building floors as well as footbridge and marine pier superstructures. The influence of partial interaction is studied by considering a low degree of shear connection and an analytical assessment of the whole response is carried out using previous formulations, highlighting a good accuracy. The behavior of the hybrid beams is further evaluated against that of equivalent reinforced concrete beams and single GFRP profiles, thus proving the feasibility of the solution.

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Highlights

- Evaluation of the structural performance of different GFRP-concrete hybrid beam models.
- Comparative study with equivalent reinforced concrete beams and pultruded profiles.
- Research on the influence of partial interaction effects and concrete strength.
- Analytical assessment of experimental results for hybrid beams with low degree of shear connection.

Keywords

Composite beam; pultruded FRP; concrete; flexural behavior; partial interaction; shear connection; analytical assessment.

1. Introduction

Pultruded fiber-reinforced polymer (PFRP) profiles have been used in the past three decades in a significant number of applications where high corrosion or chemical resistance is required and where the weight of the structure plays an important role in the design [1–3]. Structural applications have included pedestrian and road bridges, building floors and frames, stair structures, cooling towers, offshore platforms, marine piers and light support structures.

The efficiency and versatility of this relatively new construction material are a result of its outstanding mechanical, physical and chemical properties. Besides the lightweight and high strength characteristics, composite manufacturers emphasize the fact that structures built with PFRP profiles are more durable, require virtually no maintenance and can be constructed in a simple and rapid manner, without the use of extensive scaffolding. Furthermore, opposed to custom-made composites, pultruded members are produced through a lower cost fabrication process and have dimensional stability.

44 Despite their great potential, PFRP profiles also present some disadvantages when compared to their steel
45 counterparts: a relatively low stiffness (especially for glass FRP) that can lead to design constraints due to
46 instability or large deformations, an inherent brittle behavior and a partially developed connection technology.
47 In addition, the lack of codes as well as the current high initial costs of these advanced materials prevent a
48 widespread use of pultruded profiles in civil engineering projects. To overcome some of these issues,
49 researchers have proposed recently the introduction of new hybrid elements [4–6] that combine the
50 advantages of PFRP profiles with those of traditional materials so as to obtain superior structural members.

51 Most of the hybrid beams designed up-to-date have been built by combining glass fiber-reinforced polymer
52 (GFRP) profiles with concrete because of their low cost and high structural efficiency. Concrete is also
53 preferred because it can provide confinement, increase flexural stability, strength and stiffness, all at the cost
54 of an increased mass. This apparent inconvenient presents an upside in the sense that the structure will have
55 better damping, as light structures are usually prone to unacceptable vibrations.

56 The GFRP profile and concrete layer can be connected using a bonded joint, mechanical connection or
57 combined joint. Tests performed so far on hybrid beams with bonded joints have demonstrated that an
58 adhesive layer will provide a high connection strength and will practically impede the occurrence of slip,
59 providing a complete shear interaction [7–9]. Nevertheless, bonded joints require special tools, materials and
60 installment conditions, are sensitive to environmental degradation and possess insufficient ductility
61 characteristics. On the other hand, mechanical joints are easy to inspect and disassemble, have substantial
62 post-elastic capacity but, due to the flexibility and the discrete nature of the connection, partial interaction
63 effects need to be accounted for [10].

64 In the past two decades, numerous hybrid beam designs have been proposed and analyzed experimentally,
65 fueling an increased interest in this area of advanced composite materials justified by the promising results.
66 One of the first studies regarding hybrid beams was performed by Saiidi et al. [11] on graphite/epoxy concrete
67 composite beams for bridge decks and floor slabs. The investigation focused on the flexural behavior of
68 custom-made box and I-shaped profiles connected to concrete slabs with an epoxy layer, and studied the
69 composite action and the effects of concrete strength on bond, flexural stiffness and capacity. Fragile failure
70 modes were observed that consisted of shear debonding followed by longitudinal delaminations of the web.

71 Analytical calculations based on the assumption of complete shear interaction and an estimated bond strength
72 proved to be inexact. The study highlighted the need for pultruded shapes with better fiber orientation, lower
73 costs and a more accurate analytical model.

74 Sekijima et al. [12,13] investigated the behavior of GFRP-concrete beams made with H-shaped FRP profiles.
75 The shear transfer mechanism consisted of conventional studs which had been used for steel-concrete
76 composite beams, arranged in a cross stitch pattern to prevent cracking between holes. There was no buckling
77 of the hybrid specimens observed; however, the failure was sudden and occurred in the web of the profiles.
78 The experimental behavior was linearly elastic up to failure and slip between the two materials was noted.

79 Studies carried out by Biddah [14] and by Fam and Skutezky [15] analyzed the response of several hybrid
80 beams with profiles encased or filled with concrete and observed that compared to the other specimens the
81 beams displayed less deformation and slip. It was noted also that the concrete prevented local buckling of the
82 web or flanges to occur.

83 Different authors have recently proposed various solutions to improve the characteristics of the hybrid
84 systems by tailoring the properties and microstructure of the composite profiles [16], by using high
85 performance or fiber-reinforced concrete layers [17–20], or by adapting a failure sequence for the whole
86 system [21,22]. A custom hybrid profile made from CFRP and GFRP layers was designed and tested by
87 Mutsuyoshi et al. [23] in both a simple and composite configuration. The profile alone failed in flexure due to
88 delaminations at the interfacial layers and web crushing, while the composite beam performed better in every
89 aspect. Research done by El-Hacha and Chen [24] on FRP-UHPC hybrid beams and by Gonilha et al. [25] on
90 GFRP-SFRSCC elements for prototype bridge decks revealed that the increased strength of the concrete slab
91 led to a linear-elastic flexural response of the system and did not provide a failure warning, as the performance
92 was still limited by the mechanical characteristics of the composite profiles or connection.

93 Subsequently, current research indicates that there is still a great need to investigate experimentally the
94 structural behavior of pultruded FRP-concrete hybrid beams and to find solutions with lower costs.
95 Furthermore, to this point, many studies have limited their analyses by considering a state of complete shear
96 interaction although slip phenomena had been previously observed during testing.

97

98 2. Experimental program

99 2.1. Scope

100 The investigation discussed herein focuses on the analysis of the experimental structural performance of
101 hybrid beams made of pultruded glass fiber-reinforced polymer (GFRP) profiles mechanically connected to
102 reinforced concrete (RC) slabs, suitable for building floors as well as footbridge and marine pier
103 superstructures. The proposed hybrid system is designed to exploit the main advantages of its composing
104 materials whilst overcoming some of the issues that characterize their individual behavior. Thus, the GFRP
105 members are expected to carry mainly the tensile and shear forces in the composite beam, with the concrete
106 layer acting as a compressive and stabilizing top element. Commercially available profiles were used in order
107 to reduce costs and normal strength concrete was chosen so as to improve the ductility of the beams. Due to
108 the hybrid nature of the constructive system, special attention was also paid to the influence of the mechanical
109 joint between the two constitutive parts, by considering a low degree of shear connection.

110 Following the results and observations of an initial experimental campaign carried out on small-scale hybrid
111 beams with various cross-section configurations [26], a hybrid system similar to standard steel-concrete
112 composite beams was chosen as design basis for a second and more comprehensive experimental campaign
113 performed on real-scale specimens. A number of eight hybrid beams were fabricated and their flexural
114 behavior was assessed against that of equivalent reinforced concrete beams and single GFRP structural
115 profiles. The variables of the research were the type of hybrid cross-section and the concrete strength class.
116 The experimental campaign was divided in two phases depending on the specific test setup configuration and
117 observations were made regarding the short term behavior of the novel elements under positive bending
118 moments. An analytical assessment of the results in terms of capacities, deflections and internal strains and
119 stresses was also performed, highlighting a good agreement.

120

2.2. Materials

Design began with choosing an off-the-shelf glass fiber-reinforced polymer pultruded profile shape from GDP SA, France. The IPE 120 profile, classified as structural, is made from a thermosetting PR500 grade unsaturated polyester matrix (with basic formulation) reinforced with E-glass fibers. As shown in Fig. 1, the highly inhomogeneous profile is composed of unidirectional fibers which act as longitudinal reinforcement and non-woven continuous strand mats (CSM) disposed on the contour of the shape and at the center plane of the web which perform the role of shear, transverse reinforcement. The anisotropic nature of the composite material is clearly emphasized in the same figure by an electronic microscope photograph taken at the web-flange junction.

Fig. 1. GFRP structural profile: (a) cross-section structure and geometry; (b) fiber roving; (c) non-woven CSM; (d) microscopic anisotropic structure of web-flange junction.

The measured apparent density of a profile is 1.93 kg/dm^3 and the percentage of reinforcement ratting in weight lies between 50-65%. Flexural, tensile, compressive and shear properties were obtained after extensive material characterization tests performed on a minimum of 5 coupons for each test (see Fig. 2a-e). The obtained mechanical properties and corresponding standards are summarized in Table 1.

Table 1. Mechanical properties of extracted GFRP coupons: average and standard deviation values.

Mechanical property and testing method	Flexural ISO 14125	Tensile ISO 527-4	Compressive ISO 14130	
Fiber angle ^a	0°	0°	0°	90°
Strength (MPa)	734 ±36	520 ±27	406 ±30	115 ±3.1
Elastic modulus (GPa)	35.0 ±2.1	38.0 ±1.4	40.6 ±1.8	10.8 ±0.5
Ultimate strain (%)	2.10 ±0.05	1.37 ±0.11	1.02 ±0.11	1.60 ±0.13
Poisson's coefficient	0.27 ±0.02			
In-plane shear strength, ASTM D3846 ^b : 49.0 ±4.7 MPa				
Interlaminar shear strength, ISO 14130: 31.1 ±0.7 MPa				

^a Lengthwise (0°) or crosswise (90°) direction of the fibers.

^b Coupons rotated 90°.

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141 Using a linear regression analysis of the three-point bending equation which characterizes the deflection test
 142 specified in EN 13706 (Fig. 2f), the effective longitudinal and shear modulus of the profile's full section were
 143 estimated as 39.1 ± 0.1 GPa, respectively 3.98 ± 0.26 GPa. According to the manufacturer's own measurements
 144 the I-beam meets the specific performance criteria of grade E17 structural pultruded profiles [27], with
 145 reported values going as low as 207 MPa for tensile strength, 35 MPa for shear strength and 17.2 GPa for
 146 effective longitudinal modulus. These reduced values were regarded as highly conservative and may have
 147 been amended by safety coefficients, explaining thus the differences versus the experimental characterization
 148 results.

149

150 **Fig. 2.** GFRP material characterization tests: (a) flexure; (b) tension; (c) compression; (d) in-plane shear; (e)
 151 interlaminar shear; (f) full section effective moduli.

152 The second component of the tested beams, the concrete, was produced in two different compositions using a
 153 rapid hardening cement class 42.5 R, each mix corresponding to a batch of five beams. Average concrete
 154 compressive strength was determined 28 days after fabrication on cubic samples, following EN 12390. The
 155 remaining mechanical properties were obtained using the relations provided by Eurocode 2 [28] and are listed
 156 together in Table 2.

157 **Table 2.** Mechanical properties of concrete mixes: average compressive strength (f_{cm}), modulus of elasticity
 158 (E_c) and average tensile strength (f_{ctm}).

Concrete mix	Cement type and class	f_{cm} (MPa)	E_c (GPa)	f_{ctm} (MPa)
C1	CEM II/A – 42.5 R	30.0	28.6	1.90
C2	CEM II/A – 42.5 R	35.0	30.0	2.21

159

160 Steel reinforcement bars used in the fabrication of the hybrid beams were of class B500S, with a yielding
 161 strength of 500 MPa and modulus of elasticity of 200 GPa.

2.3. Proposed hybrid designs and fabrication

Two different models of hybrid GFRP-concrete beams were designed to be tested and analyzed in the investigation. Entitled M1 and M2, the hybrid models differed in the type of concrete cross-section geometry. In addition to these two, an equivalent reinforced concrete model, designated M0, was included to serve as reference in the analysis. All members had 2000 mm in length and 170 mm in height, with a top concrete slab of 400x50 mm. Fig. 3 illustrates the constructive details of the specimens.

Fig. 3. Constructive details of the tested specimens: cross-section models with top view and side view of M1 and M2 hybrid beams (mm).

The hybrid beams were all made of a GFRP profile that was attached to the bottom of a concrete slab by means of steel shear connectors. In contrast to model M2, model M1 had the profile also laterally encased in concrete, forming a T-shaped composite member. The reinforced concrete model M0 featured a similar cross-section to M1 but instead of the GFRP profile the beam had an equivalent area of steel rebars capable of

producing a theoretically similar tensile force as the profile working under partial interaction conditions. Shear connectors were installed before concreting of the hybrid beams, in pre-drilled holes located alternatively at 100 mm along the profile's upper flange, as seen in Fig. 4. M6 steel bolts with a class resistance of 8.8 and ultimate shear strength of 480 MPa were manually fastened into position with a torque of 10 Nm. The small diameter of the shanks coupled with the longitudinal alternate distribution allowed for the desired development of partial shear interaction. As a side note, for beams model M1 there was no lateral connection between the GFRP profile and the concrete, and for beams model M2 the profile's support regions were encased in 200 mm wide concrete blocks (Fig. 3, top and side views) so as to prevent a premature local crushing failure, as recommended by initial small-scale bending tests.

Fig. 4. Fabrication process for the hybrid beams: (a) installment of steel bolts; (b) completed formwork; (c) concrete casting; (d) specimens prior to instrumentation and testing.

In order to maintain the integrity of the concrete slab during transportation and testing, 5Ø8 mm steel bars were placed at its center as constructive longitudinal reinforcement. Transverse steel reinforcement was provided only at the middle and at the ends of the slab. Because the investigation focused on the flexural behavior of the beams, reference model M0 had only constructive transverse reinforcement in addition to the 3Ø12 bottom longitudinal bars. Reinforcement concrete cover was in all cases 20 mm.

Ten beams were fabricated using the three model designs: two units of model M0, four of model M1 and four of model M2. They were subsequently divided in two groups of five specimens according to the corresponding test setup.

197 2.4. Test setup and procedure

198 A month after fabrication the beams were subjected to positive moments in a three-point or four-point bending
 199 test configuration. All specimens were simply supported over a span of 1800 mm, either on elastomeric pads
 200 (test setup I) or on 40 mm wide steel cylinders (test setup II). A pair of Isolgomma 200x200x20 mm
 201 elastomeric supports with a density of 0.7 kg/dm^3 was used initially but, after the first batch of tests, results
 202 showed that this measure was too conservative taking into account that the ends of the profiles were encased
 203 in concrete.

204 In test setup I the beams were loaded at the midspan using a 250 kN MTS hydraulic actuator. A 45x20 mm
 205 wood piece was used to spread the concentrated load from the actuator head to the top of the specimen. In
 206 contrast, in test setup II the applied load produced by a 500 kN capable actuator was distributed in two parts
 207 situated approximately at a third of the span by a steel frame with semi-cylindrical supports. Loading was
 208 performed in a quasi-static manner with a constant displacement rate of 2 mm/min. Details of both test setups
 209 are illustrated in Fig. 5.

210

211 **Fig. 5.** Schematic of load arrangements and instrumentation of hybrid beams (mm).

212 Instrumentation was similar for both configurations so as to record and compare similar parameters of the
 213 flexural behavior. Deflections were measured at the midspan and at 500 mm towards each of the supports by

RIFTEK laser triangulation sensors. In the case of the beams placed on elastomeric pads the vertical displacements of the supports were registered by two Waycon LRW-M-100-S linear potentiometers. The hybrid specimens were additionally instrumented at one end with an HBM WA/20 displacement transducer (LVDT) so as to record the relative slip between the top flange of the GFRP profile and concrete slab.

Strain gauges were attached in key sections of the beams, near or at the center span and at 150 mm from one of the supports. For beams model M2 axial strains were measured across the concrete slab and the GFRP profile, in sections S1 and S2 (Fig. 5). In this way the slip strain between the two constitutive materials could be measured. In section S2 a couple of strain gauge rosettes were placed on the profile's web to determine the angular strains in the composite material. Hybrid beams model M1 were instrumented just in section S1 and along the bottom flange of the profile. The control or reference specimens, represented by the M0 reinforced concrete beams and the single GFRP profiles, were tested in similar configurations to those illustrated in Fig. 6.

Fig. 6. Laboratory setup and instrumentation: (a) Profile 2 test; (b) M2 hybrid beam in test setup I; (c) M2 hybrid beam in test setup II.

Data measured by the sensors were gathered by an HBM MGCplus data acquisition system at a rate of 50 Hz. In the case of the M2 hybrid beams a high speed camera was used to capture the development of the brittle failure at a speed of 2000 fps.

The beams were split in two groups according to the loading scheme that was applied. The characteristics of the two are outlined in Table 3 and the experimental results will be presented and discussed in the following section.

Table 3. Characteristics of test specimens.

Beam ID	Test setup	Type	Model	Weight (kN/m)	Concrete mix
M0-RCB1	I	RC	M0	1.03	C1
M1-HB1	I	GFRP-RC	M1	1.02	C1
M1-HB2	I	GFRP-RC	M1	1.02	C2
M2-HB1	I	GFRP-RC	M2	0.61	C1
M2-HB2	I	GFRP-RC	M2	0.61	C2
Profile 1	I	GFRP		0.03	
Profile 2 ^a	I	GFRP		0.03	
M0-RCB2	II	RC	M0	1.03	C2
M1-HB3	II	GFRP-RC	M1	1.02	C1
M1-HB4	II	GFRP-RC	M1	1.02	C2
M2-HB3	II	GFRP-RC	M2	0.61	C1
M2-HB4	II	GFRP-RC	M2	0.61	C2

^a Had wood block stiffeners placed at the critical sections (reaction points).

3. Results and discussion

3.1. Experimental results

3.1.1. Flexural behavior and failure modes

In order to assess the performance of the different tested beams, the structural behavior of the control specimens is discussed foremost. Beam M0-RCB1 had the typical ductile flexural response of a reinforced concrete element, failing by yielding of the bottom steel reinforcing bars accompanied by crushing of the concrete top. In the case of M0-RCB2 tested under four-point bending, the failure was sudden due to a diagonal tensile shear crack formed near one of the supports, justified by the low ratio of transverse reinforcement. GFRP Profile 1 and 2 failed both due to a loss of global stability, by lateral torsional buckling, as illustrated in Fig. 7. Consequently, after the initial failure of Profile 2, the uneven normal compressive stress provoked a local buckling of the top flange observable in the same figure. The flexural behavior of the two specimens was linear elastic, with Profile 2 attaining a higher capacity because of the stiffeners placed at the reaction points. Profile's 2 ultimate capacity was just 17% lower than that of M0-RCB1, with a maximum deflection 3 times as great.

Fig. 7. Buckling failure modes of profile control specimens: (a) Profile 1; and (b) Profile 2.

Hybrid beams M1-HB1 and M1-HB2, which were made of a GFRP structural profile encased in a T-shaped concrete beam, displayed a generally bilinear response up to ~90% of the ultimate load, a superior strength in comparison to M0-RCB1 and double the flexural rigidity of the single profiles. Furthermore, the maximum load sustained by M1-HB2 represented a threefold increase over the value recorded for Profile 1. The bilinear shape of the responses is attributed mainly to the change in the stress transfer mechanism at the connection level. Thus, the initial slope reflects a complete interaction between the two layers while the second a partial interaction (i.e., flexible connection). At the beginning of the tests, large vertical flexural cracks appeared in the concrete web of the hybrid beams due to the material's loss of tensile strength, as revealed by the jumps in the load-displacement responses represented in Fig. 8. As loading continued, the cracks progressed towards the inferior central part of the top slab. Failure of the M1 hybrid elements began with crushing of the concrete top at the midspan and ended a few moments later when the profile's bottom flange suddenly detached from the GFRP web. The cause of the brittle collapse was determined to be the increased shear stress which had developed at the web-flange junctions, at the ends of the pultruded members. After failure, the two hybrid beams continued to work in flexure, displaying a recovery capacity of up to 75% of the maximum sustained load.

Fig. 8. Experimental bending results under test setup I: load-midspan deflection curves until failure.

The flexural responses of hybrid beams M2-HB1 and M2-HB2, which were made of a GFRP structural profile attached with steel bolts to a reinforced concrete slab, were similar to those of the previous M1 hybrid beams. Slight differences are visible in Fig. 8 in the increased deformability explained by the fact that the GFRP web was not laterally encased in concrete and in the higher nonlinear response towards the end, justified by the concrete's constitutive behavior under high compressive strains. This time around the flexural cracks were less wide and more spread across the slab, starting especially from the connectors' positions and reaching towards the edges and central line. For M2-HB1 failure began with crushing of the concrete top followed by a brittle shear delamination at one of its ends, at the junction between the GFRP profile's top flange and web. The shear failure dispersed immediately towards the midspan of the beam, causing an additional vertical displacement of the steel bolts and a local buckling of the compressed web (post-failure mechanism). In contrast, failure of hybrid beam M2-HB2 occurred suddenly at the midspan, without concrete crushing, in the zone directly placed under the applied load, possibly being induced by a fracture of the load spreading piece. Thus, a high compressive stress present at the top of the GFRP profile determined a crushing type of failure to occur in the profile's web followed by longitudinal delaminations of the composite material. No significant recovery capacity was displayed by the M2 hybrid beams during the three-point bending tests.

In the second phase of the experimental campaign, the hybrid specimens tested under four-point bending exhibited a generally bilinear structural response and a higher capacity than control beam M0-RCB2. Nevertheless, the flexural stiffness was lower, with M2-HB3 and M2-HB4 experiencing a greater

289 deformability as seen in Fig. 9. The occurrence of flexural cracks is emphasized again by the sudden drops in
 290 load-bearing capacity, especially in the initial stage for the M1 beams. Opposite to the first testing phase, all
 291 hybrid beams failed in the same manner due to a longitudinal shear crack which developed at the top web-
 292 flange junction, without any prior crushing of the reinforced concrete slab. M1-HB3 and M1-HB4 retained
 293 after failure a capacity of 50-60% of the maximum load whilst beams model M2 provided inconclusive
 294 recovery results.

295

296 **Fig. 9.** Experimental bending results under test setup II: load-midspan deflection curves until failure.

297 Fig. 10 illustrates the main failure modes observed for the hybrid beams during the experimental campaign.

298 Cracks generated by the inward slip of the profile were marked in red in Fig. 10c.

299

300 **Fig. 10.** Failure modes of hybrid beams: (a) profile web-flange shear preceded by crushing of the concrete
 301 slab; (b) crushing of the profile's web; (c) profile web-flange shear.

302 Load-deflection charts reflect also the change in the slab's compressive strength, whereas beams fabricated
 303 using concrete mix C2 have a slightly higher flexural stiffness and capacity, as expected. In spite of this, the

ultimate load of the hybrid beams seems to be limited by the amount of bending deformation supported and more precisely by the amount of shear that the GFRP profile can carry. Considering a uniform distribution of the shear stress in the web of the profile and neglecting the contribution of the flanges, Fig. 11 plots the variation of the shear force percentage carried by the composite profile against the applied load.

308

309 **Fig. 11.** Hybrid beam M2-HB4, section S2: shear force carried by the profile's web in function of the applied
310 load.

311

312 3.1.2. Composite action and interlayer slip

313 Gauge measurements performed at sections S1 and S2 were used to plot the variation of the longitudinal
314 strains in function of the applied load. Fig. 12 illustrates this variation for the particular case of hybrid beam
315 M2-HB4. Similar strains across the top slab suggest that the whole width of the concrete section was effective.
316 This result is in agreement with the design code recommendations for simply supported steel-concrete
317 composite beams [29]. Negative strain values registered on the top flange of the GFRP profile indicate that the
318 pultruded element started to work in compression at higher load levels. For the specimens which failed
319 primarily due to slab crushing, concrete strain curves displayed maximum negative values in the vicinity of
320 0.35%. Maximum GFRP axial deformations were in the range of 1.1% for the beams tested under three-point
321 bending, respectively 0.6% for the specimens under four-point bending.

Fig. 12. Hybrid beam M2-HB4, section S1: variation of axial strains in function of the applied load.

The same data was used to plot the axial strains as a function of the beam's depth for different load levels. In this way, a better view of the composite action developing in the hybrid beam was obtained, exemplified here by Fig. 13 for hybrid beam M2-HB4. After 20 kN of load there was an increased slip strain developed between the concrete slab and the profile that led to the appearance of two neutral axes in the cross-section of the element. The first neutral axis of the T-shaped beam laid in the top concrete slab, close to the steel reinforcement level, while the position of the second neutral axis moved from the connection level towards the center of the composite member. Due to the relatively low elastic modulus of GFRP, shear has an important role in the behavior of short elements ($\text{height}/\text{span} < 1/20$) in the sense that at high stress levels the section does not remain plane after bending. This warping effect of the profile is slightly noticeable in Fig. 13.

Fig. 13. Hybrid beam M2-HB4, section S1: normal strain distribution at different load levels (kN).

335 Axial strain variations registered for the M2 specimens at section S2, near one of the supports, also point out
 336 that the web was in compression at higher loads; however, the top flange was still submitted to tensile
 337 deformations, being hindered by the mechanical connection. It is believed that this effect coupled with the
 338 significant in-plane shear deformation of the profile led to the web-flange shear failure of the hybrid beams.
 339 The relative slip between the profile and the slab at the end of the hybrid beams is plotted in Fig. 14 against
 340 the load ratio. Hybrid beams model M1 presented a complete shear interaction up to 40% of their ultimate
 341 flexural capacity whereas beams model M2 had a weaker shear interaction starting from about 25%. The
 342 average maximum slip was 1.7 mm for type M1 and an almost double amount of 3.5 mm for type M2.
 343 Overall, hybrid beams model M1 displayed a stiffer, higher composite action due to the concrete web which
 344 prevented the steel bolts and GFRP profile from sliding too much. In addition, the concrete class had a similar
 345 influence, with higher strengths limiting the slip to a greater degree.

346

347

Fig. 14. Relative end slip of the profile versus load ratio.

348 The slip strain-bending moment curves plotted in Fig. 15 illustrate similar nonlinear responses for the M2
 349 specimens. The exception resides in the fact that during the first testing phase the deformations attained were
 350 double in comparison with the results from the second testing phase, under four-point bending.

Fig. 15. M2 hybrid beams: slip strain variation in function of the applied bending moment.

The partial interaction effects attributed to the low degree of shear connection and flexibility of the bolts were noticed not only from numerical data but also during a visual inspection of the tested members, as illustrated in Fig. 16.

Fig. 16. Visual evidence of partial interaction: (a) occurrence of slip; (b) deformation of bolts; (c) distortion of connector holes.

3.2. Analytical assessment

Based on the analytical procedure for PFRP-RC hybrid beams detailed by the present authors in [30], the data obtained during the experimental campaign were compared with theoretical results. The procedure used is built on the Timoshenko beam theory and on the elastic interlayer slip model, where partial interaction effects over flexural capacity, deflection and strains are quantified by using a dimensionless parameter which relies mainly on the connection's stiffness and beam rigidity characteristics. The shear resistance of the hybrid

beams is considered to be entirely provided by the GFRP profile, which represents a conservative but reliable approach as suggested by the same study. For the sake of completeness, Appendix A summarizes the main mathematical relations used herein.

A comparison is made in Table 4 between the experimental and analytical results, at the serviceability and ultimate limit states (SLS and ULS). The numbers reveal a difference under 5% between the ultimate bending moments and an even smaller difference for the serviceability case. There are two larger exceptions for SLS because of the jumps in deflection measurements induced by the occurrence of large flexural cracks. The differences between maximum deflections at failure are underestimated because the concrete behavior is considered linear in the analytical model. Nevertheless, the overall stiffness of the hybrid beams considering partial interaction is estimated with good precision as it will be shown and discussed later. The ratio between the bending moment considering a shear type of failure and the one based on a compressive failure of the concrete slab, M_u/M_u^{cr} , reveals an important ductility aspect of hybrid beams. For the four-point bending test configuration the ratios are less than unity while for the three-point setup most of the values are slightly over it. This theoretical evaluation coincides with the experimental observations, where three of the eight hybrid beams, M1-HB1, M1-HB2 and M2-HB1, failed in a pseudo-ductile manner while on the contrary the rest had a predominantly fragile response.

Table 4. Results for hybrid beams at the serviceability (SLS) and ultimate limit states (ULS/u): bending moment (M), midspan deflection (w), and bottom flange ultimate axial stress (σ).

Beam	Experimental					Analytical						
	Failure mode	M_{SLS}^a (kNm)	M_u (kNm)	w_u (mm)	σ_u (MPa)	M_{SLS}^a (kNm)	diff. (%)	M_u (kNm)	diff. (%)	w_u (mm)	diff. (%)	M_u/M_u^{cr}
M1-HB1	web-flange shear ^b	7.9	36.3	42.6	420	10.5	+33.0	36.7	+1.1	35.2	-17.3	1.04
M1-HB2	web-flange shear ^b	10.4	41.5	51.5	474	10.5	+1.2	39.5	-4.8	36.5	-29.1	1.00
M2-HB1	web-flange shear ^b	10.7	35.1	52.5	406	10.5	-1.8	36.7	+4.5	35.2	-32.9	1.04
M2-HB2	web crushing	9.4	33.9	51.7	415	10.5	+12.4	33.9	+0.1	31.4	-39.4	0.86
M1-HB3	web-flange shear	9.0	21.6	23.4	210	8.8	-1.3	21.7	+0.4	25.5	+9.0	0.62
M1-HB4	web-flange shear	8.7	22.8	22.4	218	8.9	+2.2	23.4	+2.4	26.3	+17.5	0.59
M2-HB3	web-flange shear	8.8	23.9	35.2	256	8.8	+0.2	23.4	-2.3	27.5	-22.0	0.67
M2-HB4	web-flange shear	8.7	24.3	33.6	250	8.9	+1.9	23.4	-3.9	26.3	-21.6	0.59

^a Computed for a midspan deflection equal to the span/250.

^b Preceded by concrete slab crushing.

For the three hybrid specimens mentioned before, which failed initially due to concrete crushing, Table 5 presents an analytical assessment of the results considering three main hypotheses: complete shear interaction;

partial shear interaction with slip strain evaluation; and partial shear interaction using the approximate approach described in [30], based on the Eurocode 5 definition of effective flexural stiffness [31]. Differences expressed in terms of percentages indicate that the values considering complete interaction are on the unsafe side of the design, overestimating the flexural capacity of the hybrid beams, whereas results considering the third hypothesis are the most accurate.

Table 5. Flexural responses of hybrid beams considering concrete crushing: bending moment (M) and midspan deflection (w).

Beam	Experimental		Analytical									
			Complete interaction				Partial interaction					
	M_u^{cr} (kNm)	w_u^{cr} (mm)	M_u^{cr} (kNm)	diff. (%)	w_u^{cr} (mm)	diff. (%)	M_u^{cr} ^a (kNm)	diff. (%)	$M_{u,eff}^{cr}$ ^b (kNm)	diff. (%)	w_u^{cr} (mm)	diff. (%)
M1-HB1	34.2	36.5	41.1	+20.1	28.2	-22.7	36.1	+5.5	35.1	+2.6	33.7	-7.5
M1-HB2	38.8	38.6	45.3	+16.7	31.0	-19.7	39.5	+1.7	39.5	+1.8	36.5	-5.3
M2-HB1	34.9	50.2	41.1	+17.8	28.2	-43.8	36.1	+3.4	35.1	+0.6	33.7	-32.9

^a Computed by estimating the maximum interlayer slip strain.

^b Computed using the approximate approach with the dimensionless parameter.

Figs. 17 and 18 plot the analytical and experimental load-midspan displacement curves for the M1 and M2 type of hybrid beams. The structural behavior is reproduced with good accuracy by the analytical procedure, and particularly the flexural stiffness which reflects the transition from complete to partial shear interaction. The model takes into account also the cracked/uncracked state of the concrete and the bilinear behavior of the connectors, however, the small steel reinforcement contribution in the slab is neglected. The theoretical responses emulate the effects of a higher concrete strength class but limit the analysis to an elastic domain. Nonlinear behavior was more present in the M2 specimens and reflects the constitutive behavior of the concrete at higher normal stresses, before the ultimate load sustained.

Fig. 17. Hybrid beams model M1: analytical and experimental load-midspan deflection curves.

Fig. 18. Hybrid beams model M2: analytical and experimental load-midspan deflection curves.

An analytical estimation is made in Fig. 19 for the position of the neutral axes across the depth of M2-HB1. The uncracked complete shear interaction model predicts well the initial part of the variation while the cracked model with interlayer slip exhibits slight differences versus the final position of the two neutral axes before collapse.

Fig. 19. Hybrid beam M2-HB1, section S1: experimental variation of the depth of the neutral axes in function of the applied bending moment, and analytical prediction.

Experimental and numerical results for strains and stresses are compiled in Table 6. The values were calculated for an intermediate load of 50 kN, above the serviceability limit check, where the concrete's stress distribution is still plane. The percentile differences for the interlayer slip strain show that the beams had a more flexible connection than estimated. With respect to the maximum axial strain and stress which developed in the GFRP profiles, the average difference was lower, around 15%.

Table 6. Strain and stress results at intermediate load – 50 kN: interlayer slip strain at section S1 (ϵ_s), maximum GFRP axial strain (ϵ_{max}) and corresponding maximum longitudinal normal stress (σ_{max}).

Beam	Experimental			Analytical				
	ϵ_s (%)	ϵ_{max} (%)	σ_{max} (MPa)	ϵ_s (%)	diff. (%)	ϵ_{max} (%)	σ_{max} (MPa)	diff. (%)
M1-HB1		0.56	219	0.26		0.46	180	-18.1
M1-HB2		0.51	198	0.23		0.45	176	-11.1
M2-HB1	0.37	0.52	203	0.26	-28.5	0.46	180	-11.5
M2-HB2	0.38	0.54	212	0.23	-38.4	0.45	176	-16.8
M1-HB3		0.35	137	0.16		0.27	106	-22.4
M1-HB4		0.32	127	0.14		0.27	104	-17.7
M2-HB3	0.11	0.30	117	0.16	+36.8	0.27	106	-9.6
M2-HB4	0.11	0.29	113	0.14	+18.8	0.27	104	-7.8

Finally, shear stress values computed using two analytical models are plotted in Fig. 20 for M2-HB4 against the shear force carried by the hybrid beam, together with the experimental curves obtained from the strain gauge rosettes. The first model represents the formulation introduced by Gay et al. [32] which includes the longitudinal warping of the cross-section, and the second, the classic formulation of Jourawski-Collignon. A

430 significant discrepancy is noticed between the theoretical linear responses and the measured nonlinear curves,
 431 most likely explained by the anisotropic, inhomogeneous nature of the composite profile.

432
 433 **Fig. 20.** Hybrid beam M2-HB4, section S2: experimental and analytical shear stress variation computed at the
 434 position of the strain gauge rosettes, in function of the applied shear force.

435

436 4. Conclusions

437 The present study analyzed the experimental structural performance of hybrid beams made of pultruded glass
 438 fiber-reinforced polymer (GFRP) profiles mechanically connected to reinforced concrete (RC) slabs, suitable
 439 for building floors as well as footbridge and marine pier superstructures. Because the flexural behavior of a
 440 hybrid element relies greatly on the connection system, a low degree of shear interaction was considered in
 441 this work to study its effects. Lastly, a comparative analysis between experimental and analytical results was
 442 carried out. The following conclusions are drawn from the investigation:

- 443 • Glass FRP-concrete hybrid beams prove to be structurally-efficient elements with a high flexural
 444 capacity to self-weight ratio.
- 445 • Cost-effective solutions can be obtained by using off-the-shelf materials and connectors as opposed to
 446 custom-made composite systems.

- Compared to the single pultruded GFRP profiles, the hybrid beams had superior bending resistance, pseudo-ductile behavior in specific cases and no instability type of failure. Furthermore, the composite material was better used, being subjected to a stress up to 80% of its tensile strength.
- Compared to the equivalent reinforced concrete beams, the hybrid specimens displayed ~50% higher ultimate capacity with 50% less weight. Nevertheless, the flexural stiffness was lower due to the elastic modulus of the GFRP and more substantially due to the low degree of partial interaction.
- Two types of cross-section hybrid models – M1 and M2 – were considered in the experimental campaign, the difference residing in the lateral confinement of the profile for the first model. Overall, similar responses were registered for both types; however, beams M1 had a more rigid mechanical connection with slip values at half of those of M2, a more linear load-midspan deflection response and at least a 50% recovery capacity after collapse. Nonetheless, the M2 beams are more suited from a practical point of view due to the reduced self-weight. A stronger interlayer connection would compensate for the lack of lateral confinement.
- Normal strength concrete allowed for a pseudo-ductile type of failure, where crushing of the concrete slab constituted a warning sign of the imminent collapse.
- The increase in concrete strength improved the ultimate bending capacity and stiffness but the behavior was still limited by the shear deformation that the profile could bear.
- Rupture of the GFRP profile's web-flange junction constituted the primary type of failure and was produced mainly by high shear stress concentrations. As observed, the web-flange junction represents a transition area where the internal microstructure of the composite shape changes drastically and where the mid-plane multidirectional transverse reinforcement ends for the profiles used in the investigation.
- A transverse crushing failure of one of the specimens indicates that stiffeners should be placed under areas with concentrated loads to prevent a premature type of collapse. Profiles with hollow sections could also defer the occurrence of brittle failure modes as well as increase the flexural capacity.

- The analytical results matched the experimental results with good precision. Theoretical flexural responses highlighted a positive agreement in terms of bending moments, deflections and normal stresses. Furthermore, the analytical model was able to capture the influence of the partial interaction on these values.

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Appendix A. Analytical formulations

A brief description of the main mathematical expressions discussed by the authors in [30] and used in the current analytical assessment is adjoined.

The maximum deflection of a PFRP-RC hybrid beam under flexure considering a complete shear interaction behavior is expressed as a sum of the deflection due to bending and shear deformation:

$$w_{co} = \frac{f}{EI_{co}} + \frac{g}{G_p A_w} \quad (\text{A.1})$$

where f and g are functions given by the elasticity theory which depend on the load and supporting conditions; EI_{co} is the flexural rigidity of the member; G_p the shear modulus of the profile and A_w the profile's web area.

Under partial shear interaction conditions the maximum deflection of a hybrid beam is approximated as:

$$w_{pa} = (1 + \xi) \frac{f}{EI_{co}} + \frac{g}{G_p A_w} \quad (\text{A.2})$$

where ξ represents a dimensionless partial interaction parameter defined as:

$$\xi = \left(\frac{EI_{co}}{EI_0} - 1 \right) \left[1 + \left(\frac{\alpha L}{\pi} \right)^2 \right]^{-1} \quad (A.3)$$

493 The composite action parameter αL is computed using the following relationship:

$$\alpha L = \sqrt{\frac{K_c d_c^2}{s_c} \frac{EI_{co}}{EI_0 (EI_{co} - EI_0)}} L \quad (A.4)$$

494 where L represents the beam's span; EI_0 the flexural rigidity of the hybrid member under no shear interaction;
 495 K_c and s_c the connector's stiffness and spacing (pitch), respectively; and d_c the distance between the centroids
 496 of the slab and profile.

497 The flexural capacity, M , of a hybrid element is found from the equilibrium of the cross-section. For the
 498 partial interaction case the slip strain must also be determined. An approximate solution that excludes
 499 calculating the slip strain, denoted M_{eff} , is based on the partial interaction parameter ξ and is expressed as:

$$M_{eff} = M \left[1 - \xi \frac{h_p E_p}{6EI_{co}} (h_c A_p + h_p A_w) \right] \quad (A.5)$$

500 where h_p is the height of the profile; E_p the profile's flexural modulus; h_c the height of the concrete slab; and
 501 A_p the profile's transverse area.

502 The maximum axial stress evaluated at the bottom of the member is obtained from:

$$\sigma_{max} = M \left\{ \left[1 - \frac{EI_0}{EI_{co}} (1 + \xi) \right] \frac{1}{A_p d_c} + \frac{0.5 E_p h_p}{EI_{co}} (1 + \xi) \right\} \quad (A.6)$$

503 The structural capacity of a hybrid beam is also limited by the amount of shear force that the profile can carry,
 504 which is computed from:

$$V_{max} = \tau_{max} A_{sh} \quad (A.7)$$

505 where τ_{max} is the in-plane shear strength of the composite material and A_{sh} the sheared area of the profile,
 506 typically considered as the area of the web.

507

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584

585

586 **Figure captions**

587 Fig. 1. GFRP structural profile: (a) cross-section structure and geometry; (b) fiber roving; (c) non-woven
588 CSM; (d) microscopic anisotropic structure of web-flange junction.

589 Fig. 2. GFRP material characterization tests: (a) flexure; (b) tension; (c) compression; (d) in-plane shear; (e)
590 interlaminar shear; (f) full section effective moduli.

591 Fig. 3. Constructive details of the tested specimens: cross-section models with top view and side view of M1
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597 hybrid beam in test setup II.

598 Fig. 7. Buckling failure modes of profile control specimens: (a) Profile 1; and (b) Profile 2..

599 Fig. 8. Experimental bending results under test setup I: load-midspan deflection curves until failure.

600 Fig. 9. Experimental bending results under test setup II: load-midspan deflection curves until failure.

601 Fig. 10. Failure modes of hybrid beams: (a) profile web-flange shear preceded by crushing of the concrete
602 slab; (b) crushing of the profile's web; (c) profile web-flange shear.

603 Fig. 11. Hybrid beam M2-HB4, section S2: shear force carried by the profile's web in function of the applied
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608 Fig. 15. M2 hybrid beams: slip strain variation in function of the applied bending moment.

609 Fig. 16. Visual evidence of partial interaction: (a) occurrence of slip; (b) deformation of bolts; (c) distortion of
610 connector holes.

611 Fig. 17. Hybrid beams model M1: analytical and experimental load-midspan deflection curves.

612 Fig. 18. Hybrid beams model M2: analytical and experimental load-midspan deflection curves.

613 Fig. 19. Hybrid beam M2-HB1, section S1: experimental variation of the depth of the neutral axes in function
614 of the applied bending moment, and analytical prediction.

615 Fig. 20. Hybrid beam M2-HB4, section S2: experimental and analytical shear stress variation computed at the
616 position of the strain gauge rosettes, in function of the applied shear force.

617

618 Table captions

619 Table 1. Mechanical properties of extracted GFRP coupons: average and standard deviation values.

620 Table 2. Mechanical properties of concrete mixes: average compressive strength (f_{cm}), modulus of elasticity
621 (E_c) and average tensile strength (f_{ctm}).

622 Table 3. Characteristics of test specimens.

623 Table 4. Results for hybrid beams at the serviceability (SLS) and ultimate limit states (ULS/u): bending
624 moment (M), midspan deflection (w) and bottom flange ultimate axial stress (σ).

625 Table 5. Flexural responses of hybrid beams considering concrete crushing: bending moment (M) and
626 midspan deflection (w).

627 Table 6. Strain and stress results at intermediate load – 50 kN: interlayer slip strain at section S1 (ε_s),
628 maximum GFRP axial strain (ε_{max}) and corresponding maximum longitudinal normal stress (σ_{max}).