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Abstract

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Abstract: This paper presents the results of an experimental study on the bond behaviour of glass fibre reinforced polymer (GFRP) I-section embedded in concrete. A total of five specimens with the same cross-section dimension were cast and tested using push-out test. The main parameters investigated in this study were bond length (300 mm and 450 mm), transverse stirrups and sand coating. The experimental results show that the ultimate bond stress can be improved by a longer bond length and sand coating. However, the ultimate bond stress was reduced when stirrups were used, and the reason may be because the application of stirrups affected the vibration of the concrete, causing a weak bond at the interface. The bond stress distribution at the web and the flange is analysed based on the strain of the I-section. Finally, a bond stress-slip model is proposed for the GFRP I-section with a smooth surface. Based on this model, the theoretical results are in good agreement with the experimental results.

Keywords: Bond-slip; Theoretical model; GFRP pultruded profile; I-section; Concrete.

46 Research Highlights

47 Bond behaviour between GFRP I-section and concrete is assessed.

48 Push-out test is used to investigate the bond behaviour of GFRP I-section to concrete.

49 Effect of bond length and sand coating is investigated.

50 Theoretical model of bond stress-slip relationship is proposed.

1. Introduction

Fibre Reinforced Polymer (FRP) pultruded profiles have been increasingly investigated in recent years. Conventional FRP Pultruded profiles are usually made of glass fibres embedded in a vinylester or polyester matrix (GFRP Pultruded profiles) [1, 2], and have some advantages such as low self-weight, high strength and corrosion resistance [3]. Moreover, GFRP pultruded profiles are recommended to be used in projects with a demand for faster construction due to ease of installation [4]. The application of GFRP pultruded profiles mainly includes two types, all GFRP structures [5] as well as hybrid structures [6, 7]. Among them, the hybrid structures reinforced with GFRP pultruded profiles have recently gained more attention due to the superior structural behaviour [8, 9].

The performance of hybrid structures is dependent upon the properties of concrete and reinforcement, as well as the bond behaviour between the two components [10]. Therefore, an adequate bond between the concrete and the reinforcement is important for the performance improvement of hybrid structures. Nevertheless, the bond behaviour of GFRP pultruded profiles in concrete is traditionally weak due to the smooth surface, thus causing a poor performance of the hybrid structures [7, 11-13]. Moreover, when compared with GFRP bars or steel bars, GFRP pultruded profiles usually have a larger surface. Hence, the influence of the bond behaviour at the interface is more significant. In order to achieve a good composite action for the GFRP pultruded profiles used in the composite structures, it is essential to understand the bond mechanisms and determine the bond-slip constitutive laws.

The existing investigation of bond-slip model of FRP can be divided into two types, FRP

74 sheet/plates bonded to concrete [14-17] and FRP bars in concrete [10, 18, 19]. These two
75 types of bond-slip models are not suitable for the GFRP pultruded profiles. The bond-slip
76 model for FRP sheet/plate bonded to concrete cannot be used for GFRP pultruded profile due
77 to the different interface properties. Epoxy resin is usually used between FRP sheet/plate and
78 concrete to provide strong adhesion, while no adhesive is used to bond the GFRP pultruded
79 profiles and concrete. In terms of FRP bars, the size effect cannot be ignored since the
80 majority of GFRP pultruded profiles have a much larger surface (i.e. GFRP I-section, GFRP
81 tube) than FRP bars. Therefore, the investigation on the bond behaviour of the GFRP
82 pultruded profiles in concrete is needed both experimentally and theoretically.

83
84 The bond behaviour of GFRP pultruded profiles in concrete was firstly investigated in this
85 experimental study. The GFRP pultruded profiles used was GFRP I-section (I-section). For
86 the experimental method, the common pull-out test [20, 21] was not used due to the difficulty
87 of fixing the I-section in the testing machine, and a push-out test [22, 23] was adopted in this
88 study. As a preliminary test, a total of five specimens with different configurations were
89 tested. The parameters investigated included bond length, transverse stirrups and sand
90 coating. Based on the experimental results, the failure modes, bond stress-slip curve and bond
91 stress distribution are presented. Afterwards, the effect of stirrups and bond length is
92 discussed, and the mechanism of the load transfer along the interface between the I-section
93 and the concrete is analysed. Finally, a bond stress-slip constitutive model is proposed. The
94 predictions from this model are in close agreement with the experimental results.

2. Experimental Program

2.1 Test Specimens

A total of five specimens (Fig. 1) were fabricated and tested, and Fig. 2 shows two types of cross-section for the five specimens, Section A-A (Specimens A and B) and Section B-B (Specimens AS, BS and BSS). The design of the specimens, including the dimensions of the cross-section as well as the space between the stirrups, has been justified by a flexural test conducted by the authors [24]. Both types of cross-section have dimensions of 200 mm in width and 350 mm in length. For all the specimens, the I-sections were placed at the centre of the concrete, and the web was parallel with the long side of the cross-section. At the top end of each specimen, part of the I-section (free end) was left outside of the concrete to push the I-section out.

A 50 mm clear distance was left for the debonding at the bottom of the I-section, and a layer of plastic tape was employed on the surface of the I-section to debond the concrete and the I-section within this region (debonding region). As shown in Fig. 3, the design of this debonding region refers to the design of the specimens for the pull-out test of FRP bars as recommended by ACI 440.3R-04 [25]. This debonding region is beneficial for the push-out of GFRP I-section without the effect of the crushing of the concrete.

The R10 steel bars with 10 mm nominal diameter and 250 MPa nominal tensile strength were used as stirrups in Specimens AS, BS and BSS. Since the longitudinal bars cannot provide any confinement for the concrete, it is believed that these bars have little effect on the bond behaviour. Therefore, R10 bars were also employed as longitudinal reinforcement for ease

fabrication of the specimens. Table 1 shows the test matrix of the specimens.

The label of the specimens consists of three parts. The first part is the letter A or B, which indicates the different bond length of the specimen (300 mm for A and 450 mm for B). The second part is the letter S indicating that the transverse stirrups are used in this specimen. Lastly, the third letter S in the label means that sand coating was used on the surface of the I-section.

Specimen A was made of the I-section and concrete as shown in Fig. 1a, and the bond length is 300 mm. Transverse stirrups were used in Specimen AS to investigate the effect of stirrups on improving the bond behaviour (Fig. 1b), and four longitudinal bars were used to fix the transverse stirrups. Specimen B (Fig. 1c) was composed of the concrete and the I-section, and the bond length is 450 mm. The longitudinal bars and transverse stirrups were used in Specimen BS and Specimen BSS (Fig. 1d). Moreover, the I-section in Specimen BSS was coated with sand to improve the friction at the interface.

2.2 Material Properties

Five samples of R10 steel bars were tested in tension based on the AS 1391 [26]. The average tensile yield strength of the steel bars was 309 MPa and the elastic modulus was 192.5 GPa. The concrete was supplied by a local company with a nominal compressive strength of 30 MPa and a slump of 120 mm, and the main composition of the concrete is given in Table 2. Three concrete cylinders (100 mm $\times$ 200 mm/diameter $\times$ height) were cast to determine the compressive strength of concrete. The cylinders were tested at 28 days and the average

compressive strength of concrete was 31.8 MPa.

Fig. 4 shows the I-section used in this study, which was provided by Treadwell Group Company [27] and manufactured by a pultrusion technology. The dimensions of the I-section were 10 mm in thickness (both in flange and web), 100 mm in width and 200 mm in height. Traditionally, the majority of GFRP fibres in the pultruded profiles are laid in the longitudinal direction, therefore, only longitudinal strength of both web and flange were determined. The tensile tests were conducted by using ISO 527 [28] and the compressive strength was determined using ASTM D695 [29]. In total 20 coupons extracted from the I-section were tested. Ten coupons (five from flange and five from web) were tested to determine the tensile strength and the other ten coupons (five from flange and five from web) to determine the compressive strength of the I-section. The coupons for the tensile strength test had nominal dimensions of 25 mm in width and 250 mm in length, and for the compressive strength test, the nominal dimensions of the coupon were 12.7 mm in width and 38.1 mm in length. The test results are summarized in Table 3.

2.3 Preparation of Specimens

The preparation process of the specimens included attaching the strain gages and casting concrete. Strain gages were first attached at the longitudinal direction of the flanges and webs, and all the strain gages were set up within the bond region as shown in Fig. 5. A total of 10 strain gages was attached at the I-section of Specimens A and AS, five strain gages (S1 - S5) at the flanges and five (S6 - S10) at the web (Fig. 5a). For Specimens B, BS and BSS, seven strain gages (S11 - S17) were attached at the flange and the other seven strain gages

(S18 – S24) at the web (Fig. 5b).

Afterwards, the I-section attached with strain gages was placed into the timber formwork. In order to fix the I-section at the centre of the formwork, two tiny holes were drilled into the bottom of the formwork as well as the bottom of the I-section. All the holes were 10 mm in depth. Afterwards, two 20 mm long thin steel wires were inserted into the holes of the I-section and the formwork to fix the I-section in the formwork (Fig. 6a), and the steel wires were removed from the I-section before the test. No concrete cover was left at the bottom of the specimens. After the I-section was fixed in the formwork, the steel cage was placed into the formwork. Two steel wires with the same length as the cross-section of the specimens were used to ensure the accurate location of the steel cage, and these two steel wires were fixed at the top stirrup in the transvers and longitudinal directions, respectively (Fig. 6b).

Vibration was carried out when the concrete was cast. In order to keep the moisture, a wet hessian was placed over the specimens and the specimens were watered every day. After seven days, the specimens were demolded (Fig. 7) and then cured in moist conditions until the test day.

2.4 Test Setup and Instrumentation

The push-out test was conducted using the 5000 kN testing machine. As shown in Fig. 8, the specimen was vertically placed onto the testing machine. One steel plate was horizontally placed at the top of the I-section to uniformly distribute the load. Two steel blocks were placed under the bottom of the specimen, and adequate space under the specimen was left for

the slip of the I-section. The displacement of the loaded end were measured using two Linear Variable Differential Transformers (LVDTs), which were set up at the corners between the loading plates and supporting steel plate. In order to measure the displacement of the unloaded end, one LVDT was vertically placed under the specimen. The loaded end and unloaded end in this study refer to the ends of the I-section. The load and displacement data were recorded by an electronic data-logger connected to a computer every 2 seconds. After all these setups were completed, the specimens were loaded by a displacement controlled load with a rate of 0.1 mm/min. When the I-section was pushed out and the load did not increase, the test was terminated.

3. Experimental Results

The average bond stress (τ) in this study is defined by:

$$\tau = \frac{P}{LC} \quad (\text{Eq. 1})$$

where P is the applied load at the loaded end, L is the bond length of the I-section and C is the perimeter of the I-section.

The slip (S) in this study is defined as the relative slip between the I-section and the concrete at the loaded end. Before the I-section is pushed out, the displacement of the unloaded end is the vertical extension of the specimen based on the experimental results, which is explained in the following discussion section. Therefore, the slip (S) before pushing out the I-section is calculated taking into account the vertical extension of the specimen as below:

$$S = \Delta_1 - \Delta_2 \quad (\text{Eq. 2a})$$

where Δ_1 is the displacement of the loaded end and Δ_2 is the displacement of the unloaded

end.

After the I-section is pushed out, the displacement of the loaded end (Δ_1) and the unloaded end (Δ_2) keep same increment, and the displacement of the unloaded end (Δ_2) does not represent the vertical extension of the specimen any more. Therefore, the slip (S) is equal to the displacement of the loaded end (Δ_1):

$$S = \Delta_1 \quad (\text{Eq. 2b})$$

3.1 Failure Modes

Five specimens were cast and tested, and the failure modes are shown in Fig. 9. The I-sections in the four specimens (Specimens A, AS, B, BS) were pushed out. The surface of the I-section was intact after the I-section was pushed out, which indicates that the shear failure occurred on the interface between the concrete and the I-section. Few cracks were observed on the concrete of Specimens A and B, while the development of cracks was delayed in Specimens AS and BS due to the application of the stirrups. The I-section in Specimen BSS could not be pushed out, and this I-section failed due to the premature compressive failure at the loaded end (Fig. 10).

3.2 Bond Stress-slip Curves

The bond stress-slip curves of four specimens (A, AS, B, BS) are shown in Fig. 11a, and the typical curve is shown in Fig. 11b. In the first branch (O-A), the initial bond stress increased slowly. Afterwards, an almost linear increase of the bond stress was revealed from Point A to the ultimate bond stress (τ_s) at Point B with a larger slope. After Point B, the bond stress

curve experienced a slight decrease to Point C, and then increased again to Point D where the largest bond stress is reached. It should be noted that the ultimate bond stress (τ_s) is obtained at Point B rather than Point D in this study, and the explanation of this is given in the sections below. A descending branch could be observed after Point D. Finally, the slip showed a stable increase and the residual bond stress (τ_r) of the four specimens almost remained constant within 0.3-0.4 MPa. The experimental results of all the specimens are summarized in Table 4, including the ultimate bond stress (τ_s), the residual bond stress (τ_r) as well as the ultimate slip (S_s) and the residual slip (S_r).

The bond stress-slip curve of Specimen BSS is shown in Fig. 11c, which has a different stress-slip response compared with the other four Specimens (A, AS, B, BS). After the fluctuation in the initial stage, the curve increased linearly to the maximum bond stress where the premature failure of the I-section occurred. The largest bond stress among the five specimens was observed in Specimen BSS. For all the specimens, the slip occurred inside the specimen thus causing limited experimental observation, so the in-depth interpretation of the bond stress-slip curves could not be given based on the experimental observation only. More explanation about these curves is presented accompanied with the analysis of the strain of the I-section in the following parts.

3.3 Strain distribution of the I-section

The strain distribution taken from the strain gages at the flange and web of the I-section is shown in Fig. 12 and Fig. 13. Due to the similarity, the strain distribution of the I-section in Specimen A (Fig. 12) is analysed as a typical strain distribution for Specimen A and

Specimen AS, and the strain distribution of Specimen B (Fig. 13) is the typical distribution for the Specimen B and Specimen BS. The strain distribution of the I-section in Specimen BSS is not discussed in this study due to the premature failure at the loaded end of the I-section.

Fig. 12a and Fig 13a show the strain-load curves of the flange. All the strain increased with the increase of the load. In general, the strain near the loaded end showed a more significant increase than the strain near the unloaded end, the reason for this may be that the applied load had been counteracted by the bond stress near the loaded end. Therefore, the load had little effect on the unloaded end thus causing a small strain. However, it was observed that the strain of S2 (or S12) was larger than that of S1 (or S11) at the flange, the reason of which may be the stress concentration at the position of S2 (or S12). Since when the specimens were loaded, the compressive force at the loaded end may have caused the expansion of the web of the I-section, thus further resulting in a stress concentration to occur at the flange, in the position of strain gages S2 (or S12). Therefore, the strain of S2 (S12) was abnormally higher than S1 (or S11) in all of the specimens. The strain distribution along the flange under different load is shown in Fig. 12b and Fig 13b. The similar strain-load curves and the strain distribution were observed at the web for Specimen A (Fig. 12c and Fig. 12d) and Specimen B (Fig. 13c and Fig. 13d).

3.4 Bond stress distribution of the I-section

The bond stress distribution along the I-section is also studied based on the strain difference between two strain gages. As shown in Fig. 14, the local bond force between two adjacent

cross-sections could be calculated by:

$$\sigma_a A_s - \sigma_b A_s = \tau_1 dx C \quad (\text{Eq. 3})$$

where σ_a and σ_b are the stress at two adjacent cross-sections of the I-section; A_s is the cross-sectional area of the I-section; τ_1 is the local bond stress between two adjacent cross-sections; dx is the length between two adjacent cross-sections.

The stress σ_a and σ_b could be calculated by the corresponding elastic modulus (E) and the compressive strain (ε_a and ε_b), so the local bond stress (τ_1) is calculated by:

$$\tau_1 = \frac{EA_s(\varepsilon_a - \varepsilon_b)}{dx C} \quad (\text{Eq. 4})$$

It is noted that the elastic modulus (E) and the compressive strain (ε_a and ε_b) were experimentally determined in this study, therefore, these parameters were easily influenced by the technical problems or the testing machine, thus affecting the accuracy of the calculation for the local bond stress (τ_1). As a result, the local bond stress (τ_1) determined by Eq. 4 was employed only for the investigation of the bond stress distribution in this study. The comparison between the local bond stress and the average bond stress determined by Eq. 1 is given in Fig. 14b.

The bond stress distribution at the flange and the web for Specimen A and Specimen B is shown in Fig. 15. It is clear that the bond stress distribution is not uniform along the flange or web. In the initial stage of the test, the majority of the bond stress was distributed near the loaded end, and it was small near the unloaded end. As the increase of the load, the bond stress near the unloaded end was gradually increased until the failure of the specimen.

4. Discussion and Analysis

4.1 Slip process

The interaction between the I-section and concrete is similar to steel-concrete composite systems, so partial-interaction theory [30] could be referred to develop an in-depth mechanical analysis for the interaction between two elements. Based on the analysis of the strain and bond stress distribution as above-mentioned, the preliminary analysis about the slip process of the I-section is presented in Fig. 16. Traditionally, the mechanics of stress transfer by the bond between reinforcements (e.g., Steel/FRP bars) and concrete is mainly controlled by three factors [10, 31]: (a) chemical adhesion provided by the concrete; (b) friction due to the roughness of the reinforcements; (c) mechanical interlocking offered by the deformation of the reinforcements. The three mechanisms are not isolated during the slip process, and each mechanism has different performance in the different stages of test. The surface of the I-section is traditionally smooth, therefore, mechanical interlocking is ignored in this experimental study, and only chemical adhesion as well as friction is considered.

In this study, the interface between the I-section and the concrete was divided into two regions, bond region and slip region. The interface in bond region is intact without slip, and the bond force in bond region was dependent upon both chemical adhesion and friction. In slip region, the chemical adhesion was degraded due to the slip at the interface, therefore, only friction was contributed to the bond. The letters in Fig. 16 indicate the different stages of the test, which have the same meaning as the letters in Fig. 11b.

When the I-section was loaded in the initial stage (O-A), the entire interface between the

concrete and the I-section was bond region which provided the bond force to counteract the applied load. Afterwards, the different deformation between the concrete and the I-section at the loaded end was increased with the increment of the load, thus causing a sudden relative slip at the interface. Therefore, the slip region occurred at the loaded end of the specimen, and it was also the reason why a fluctuation of the bond stress-slip curve at Point A (Fig. 11b) was observed. When the bond stress reached the ultimate bond stress (Point B), the I-section could not provide larger bond stress, therefore, the forces were unbalanced and the original interface was totally broken. The slip region was extended to the entire interface (Loading stage B-C) with a slight drop at the bond stress-slip curves, and I-section was pushed out at the same time (Point B).

The sudden slip at Point B caused a new interface which had a coarse surface. This new interface could provide a larger friction to balance the applied load. Hence, the applied load increased again from Point C to Point D. Although maximum stress was observed at Point D, this stress could not reflect the bond behaviour of the original interface due to the damage of the interface at Point B. With the increase of the slip, the interface was smoothed and the friction was decreased. Finally, the load and the friction force reached the equilibrium state again, and the I-section was gradually pushed out.

4.2 Effect of stirrups, bond length and sand coating

Based on the analysis of the experimental results and the failure modes, it is clear that the application of stirrups did not improve the bond strength as expected. As shown in Fig. 17, the ultimate bond stress (τ_s) is decreased by using the stirrups, the possible reason for this

might be that the application of stirrups affected the vibration of concrete during the casting, thus causing a decrease of the bond strength at the interface of the I-section. The development of cracks was reduced by the stirrups in Specimens AS, BS and BSS.

The influence of the bond length was investigated by comparing the specimens with different bond length (Fig. 18). For specimens with the same bond length, the same initial stiffness was observed even though stirrups were used in one of the specimens (Fig. 17). For specimens with different bond length, the ultimate bond stress (τ_s) of the specimen was improved by the longer bond length. For example, the ultimate bond stress (τ_s) was increased from 0.46 MPa in Specimen A to 0.51 MPa in Specimen B due to the increase of the bond length.

The I-section in Specimen BSS was coated with sand to investigate the influence of sand coating on the bond behaviour. Nevertheless, the I-section crushed at the loaded end and could not be pushed out. Although the accurate ultimate bond stress (τ_s) could not be obtained in Specimen BSS, the bond stress in Specimen BSS had exceeded more than 1.3 MPa, which had been more than two times the ultimate bond stress of the I-sections without sand coating. Therefore, the bond strength could be significantly improved by using sand coating. More tests should be conducted to accurately estimate the influence of sand coating on the bond stress.

4.3 Theoretical Modelling

In this study, only the initial ascending stage (Stage O-B) of the bond stress-slip curves was investigated. The main reasons for this include: (a) the I-section was pushed out at Point B

(Fig. 11b), therefore, Stage O-B can accurately reflect the bond behaviour of the original interface of the specimens; (b) the randomness of the descending stage from B to C could not be accurately predicted; (c) after the I-section was pushed out after Point B, the bond behaviour of the interface is obviously different from the original interface.

As the material properties of GFRP bars is similar to the I-section, the bond stress-slip relationship of GFRP bars in concrete is investigated to understand the bond behaviour of the I-section in concrete. Several bond stress-slip constitutive models for FRP bars have been reported and summarized in Table 5. Among these models, the BPE model proposed by Eligehausen et al. [32] is the classical model. To start with, this model was applied to the bond of steel bars to concrete, and then successfully used for the bond behaviour of FRP bars to concrete by Rossetti et al. [33]. The bond stress-slip curve in this model is divided into different parts based on some representative parameters, such as the ultimate bond stress (τ_s), the ultimate slip (S_s) and the parameters s_f , s_r , α and β .

Using curve fitting on the experimental results, the parameter α in this model was determined as 2.5. Therefore, the bond stress-slip relationship in the curvilinear ascending branch is proposed as:

$$\tau = \tau_s \left(\frac{s}{S_s} \right)^{2.5} \quad (0 < s \leq S_s) \quad \text{Eq. 5}$$

where S is the slip at the loaded end and τ is the average bond stress. The experimental results of the ultimate bond stress (τ_s) and ultimate slip (S_s) were used in this calculation. The comparison between the theoretical model and the experimental results are presented in Fig. 19. A good agreement is observed in the ascending branch for the four specimens, especially

in Stage A-B.

The prediction of the bond stress in Eq. 5 requires the given ultimate bond stress (τ_s) and the corresponding loaded end slip (S_s). For GFRP bars, some empirical equations were proposed to obtain these two parameters. Nevertheless, in this experimental study, the number of specimens was not sufficient for an accurate empirical model to predict these two parameters. Therefore, more studies should be conducted to estimate the ultimate bond stress (τ_s) and the corresponding loaded end slip (S_s).

5. Conclusion

In this investigation, the experimental results and the bond stress-slip model on bond behaviour of the GFRP I-section in concrete were reported. Five specimens with different configurations were tested using push-out test. Based on the experimental results, the following conclusions are drawn:

1. Push-out test is an effective method to investigate the bond behaviour of the GFRP pultruded profiles in concrete.
2. The ultimate bond stress is improved by longer bond length and by using sand coating. Although the I-section with sand-coating could not be pushed out, the larger bond stress of this specimen had proved that sand coating is an effective measure to improve the bond strength.
3. The ultimate bond stress was reduced when using stirrups to confine the concrete, the reason may be because the stirrups affected the vibration of concrete, causing weak bond at

the interface between the I-section and the concrete.

4. The bond stress distribution at the web and flange was investigated based on the strain of the GFRP I-section, and two components showed similar bond stress distribution. The bond stress performed a nonuniform distribution and is mostly distributed in the loaded end.

5. An empirical model was proposed to predict the curvilinear ascending branch of the bond stress-slip curve. The results of the proposed model were in good agreement with the experimental results. Nevertheless, this model is based on the ultimate bond stress (τ_s) and the corresponding loaded end slip (S_s), therefore, a method for predicting these two parameters needs to be established.

As a preliminary experimental study, this study provides a significant reference for investigating the bond behaviour of the I-section or other pultruded profiles with respect to the test method (push-out test) and the design of the specimens. More variables should be investigated such as the compressive strength and the type of concrete, as well as the shape of the profiles, thus developing a more accurate bond stress-slip model. Moreover, the study in future should focus on improving the bond strength by using sand coating or other roughening treatment due to the small ultimate bond stress.

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Table 1. Configuration of Specimens

Group	Specimen	Cross-Section (mm)	Total height (mm)	Height of free end ¹ (mm)	Bond length ² (mm)	Height of debonding region (mm)	GFRP I-section (mm)	Stirrups (mm)	Longitudinal bars (mm)	Surface of the I-section
Group A	A	350×200	400	50	300	50	200×100×10	-	-	Smooth
	AS	350×200	400	50	300	50	200×100×10	Steel R10 @ 60	Steel 4 R10	Smooth
Group B	B	350×200	600	100	450	50	200×100×10	-	-	Smooth
	BS	350×200	600	100	450	50	200×100×10	Steel R10 @ 60	Steel 4 R10	Smooth
	BSS	350×200	600	100	450	50	200×100×10	Steel R10 @ 60	Steel 4 R10	Sand coated

¹Free end is the part of the I-section out of the concrete.

²Bond length = height of the concrete – height of debonding region (See Fig. 1 for the details)

Table 2. Composition of concrete

Constituent (kg/m ³)	Values
Cement	285
Fly ash	100
Coarse aggregate	1135
Coarse sand	543
Fine sand	217
Water	170

Table 3. Material Properties of GFRP I-section

Position	Dimensions of coupon (mm)	Property	Averages and Sample Standard Deviations
Flange	25 × 250	Tensile strength (MPa)	381.5 ± 8.1
		Tensile elastic modulus (GPa)	38.5 ± 4.2
	12.7 × 37.1	Compressive strength (MPa)	214.2 ± 17.4
		Compressive elastic modulus (GPa)	26.9 ± 1.5
Web	25 × 250	Tensile strength (MPa)	353 ± 30
		Tensile elastic modulus (GPa)	32.88 ± 1.8
	12.7 × 37.1	Compressive strength (MPa)	233.8 ± 18.4
		Compressive elastic modulus (GPa)	30.2 ± 8.5

Note: Tensile properties were determined based on ISO 527 (1997); Compressive properties were determined based on ASTM D695 (2002).

Table 4. Experimental Results

Group	Specimen	Ultimate bond load (P_s) ¹ (kN)	Ultimate load (P_u) ² (kN)	Ultimate bond stress (τ_s) (MPa)	Residual bond stress (τ_r) (MPa)	Ultimate slip (S_s) (mm)	Residual slip (S_r) (mm)
Group A	A	109.8	116.6	0.46	0.32	1.09	4.35
	AS	72.1	99.7	0.30	0.29	1.04	4.45
Group B	B	184.6	193.5	0.51	0.36	1.61	5.02
	BS	122.2	138.1	0.34	0.25	1.40	4.74
	BSS	-	474.2	-	-	-	-

¹Ultimate bond load is reached at Point B as shown in Fig. 11b.

²Ultimate load is reached at Point D as shown in Fig. 11b.

596 **Table 5. Existing bond–slip models for FRP bars**

Model	Ascending branch	Descending branch	shapes of curves	parameters
Malvar model [34]	$\frac{\tau}{\tau_s} = \frac{F \left(\frac{S}{S_s} \right) + (G - 1) \left(\frac{S}{S_s} \right)^2}{1 + (F - 2) \left(\frac{S}{S_s} \right) + G \left(\frac{S}{S_s} \right)^2}$ $\frac{\tau_s}{f_t} = A + B \left[1 - \exp\left(-\frac{C\delta_r}{f_t}\right) \right]; S_s = D + E\delta_r$			A,B,C,D,E,G = empirical constants determined for each bar type δ_r = confining axisymmetric radial pressure f_t = tensile concrete strength
Eligehausen et al. model (BPE model) [32] ^a	$\tau = \tau_s \left(\frac{S}{S_s} \right)^\alpha$	$\tau = \tau_s - (\tau_s - \tau_r) \left(\frac{S - S_f}{S_r - S_f} \right)$ $\tau_r = \beta \tau_s$		α, β = curve-fitting parameter
BPE modified model [10] ^a	$\tau = \tau_s \left(\frac{S}{S_s} \right)^\alpha$	$\tau = \tau_s \left[1 - p \left(\frac{S}{S_s} - 1 \right) \right]$ $\tau_r = \beta \tau_s$		α, β, p = curve-fitting parameter
Zhang et al. model [35] ^a	$\tau = \tau_s \left[1 - \left(\frac{S}{S_s} - 1 \right)^2 \right]$	$\tau = \tau_s - (\tau_s - \tau_r) \left(\frac{S - S_f}{S_r - S_f} \right)$	as above	-
CMR model [36] ^a	$\tau = \tau_s \left[1 - \exp\left(-\frac{S}{S_s}\right) \right]^\alpha$	-		α = curve-fitting parameter
Tighiouart et al. [37] ^a	$\tau = \tau_s [1 - \exp(4s)]^{0.5}$	-	as above	-

597 ^aThe values of τ_s , S_s , S_f , τ_r , S_r was calibrated on the basis of the experimental results.

Fig. 1. Schematic diagram of specimens (mm)
(See Fig. 2 for sections A-A and B-B)

(a) Cross-section of Specimens A and B (Section A-A)

(b) Cross-section of Specimens AS, BS and BSS (Section B-B)

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(b) Strain distribution along flange

(c) Strain-load curves of web

(d) Strain distribution along web

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(a) Strain-load curves of flange

(b) Strain distribution along flange

(c) Strain-load curves of web

(d) Strain distribution along web

Fig. 13. Analysis of strain (Specimen B)

(a) Calculation of local bond stress

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