Flexural behaviour of composite beams reinforced with GFRP I-section

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Flexural behaviour of composite beams reinforced with GFRP I-section

A thesis submitted in partial fulfilment of the requirements for the award of the degree

Doctor of Philosophy

From

University of Wollongong, Australia

By

Jiansong Yuan

School of Civil, Mining and Environmental Engineering

Faculty of Engineering and Information Sciences

July 2017
Dedicated to

My beloved parents and wife
DECLARATION

I, Jiansong Yuan, declare that this thesis, submitted in fulfilment of the requirements for the award of Doctor of Philosophy, in the School of Civil, Mining and Environmental Engineering, University of Wollongong, Australia, is wholly my own work unless otherwise referenced or acknowledged, and has not been submitted for qualifications at any other university or academic institution.

Jiansong Yuan

July 2017
ACKNOWLEDGEMENTS

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PUBLICATIONS

Journal papers:


Conference paper:

ABSTRACT

Fibre reinforced polymer (FRP) pultruded profiles have found increasingly wide applications in recent years in civil engineering. Compared with the traditional FRP composites (e.g. FRP sheet or FRP bars), FRP pultruded profiles have some distinct advantages, such as the tailorability of the cross-section and ease of installation, which are desirable features in practical implementation. This thesis presents a research study on the application of FRP pultruded profiles in the composite beams, as well as the bond behaviour of the FRP pultruded profiles to concrete. The FRP pultruded profiles used in this study is the glass fibre reinforced polymer (GFRP) I-section.

In order to improve the ductility of the composite beams reinforced with FRP pultruded profiles, a new type of composite beams reinforced with FRP I-section and longitudinal tensile steel bars were proposed in this study. A total of five beam specimens were cast and tested by using four-point bending to investigate the flexural behaviour. The parameters included the location of the I-section and the type of the tensile bars. The test results show that the proposed composite beams possess a very ductile response due to the existence of the tensile steel bars, and the yield point of the composite beam was controlled by the tensile steel bars. Moreover,
the ultimate load of the proposed composite beam was higher than the traditional reinforced concrete (RC) beam, and the ultimate load was governed by the encased I-section. The different location of the I-section in the cross-section had little effect on the flexural response of the beam specimens.

The relative slip between the concrete and the I-section was revealed in the flexural test, which affected the flexural response of the composite beams. Therefore, a push-out test was then conducted to investigate the bond behaviour between the I-section and the concrete. The specimens for the push-out test were in the form of a rectangular column with an I-section encased in concrete, and had the same cross-section dimensions as the beam specimens. The experimental results show that the ultimate bond strength could be improved by a longer bond length and sand coating. However, when stirrups were used, the ultimate bond strength was reduced. Then, a preliminary bond stress-slip model was proposed and the theoretical results were in good agreement with the experimental results.

The push-out test was followed by a direct shear test to determine the friction coefficient between the I-section and concrete. As a significant parameter of the interface, the friction coefficient cannot be determined by the push-out test, so a direct shear test was adopted in the study to obtain this parameter. The specimens
were composed of a concrete block and a coupon of the I-section. The variables investigated included the type of the concrete, the coupons from a different part of the I-section and the compressive strength of the concrete. The test results show that the compressive strength of the concrete and the different component of the I-section had little effect on the friction coefficient, while the type of the concrete significantly affected the friction coefficient. The friction coefficient between the concrete and the I-section was between 0.5 and 0.6, and the adhesion stress was approximately 0.2 MPa.
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<th>Symbol</th>
<th>Description</th>
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<tr>
<td>$a$</td>
<td>distance from the support to the loading point</td>
</tr>
<tr>
<td>$A_s$</td>
<td>cross-sectional area of I-section</td>
</tr>
<tr>
<td>$c$</td>
<td>adhesion stress between concrete and I-section coupons</td>
</tr>
<tr>
<td>$C$</td>
<td>perimeter of I-section</td>
</tr>
<tr>
<td>$d$</td>
<td>diameter of column specimens</td>
</tr>
<tr>
<td>$dx$</td>
<td>length between two adjacent cross-sections of I-section</td>
</tr>
<tr>
<td>$E$</td>
<td>compressive elastic modulus of I-section</td>
</tr>
<tr>
<td>$E_E$</td>
<td>elastic energy</td>
</tr>
<tr>
<td>$E_{frp}$</td>
<td>elastic modulus of FRP sheets</td>
</tr>
<tr>
<td>$EI$</td>
<td>bending stiffness of beam specimens</td>
</tr>
<tr>
<td>$E_T$</td>
<td>total energy calculated based on the area under load-midspan</td>
</tr>
<tr>
<td>$f_{FRP}$</td>
<td>tensile strength of the FRP sheet in hoop direction</td>
</tr>
<tr>
<td>$f_l$</td>
<td>lateral confining pressure</td>
</tr>
<tr>
<td>$f_t$</td>
<td>tensile concrete strength of FRP bars</td>
</tr>
<tr>
<td>$H$</td>
<td>height of the columns specimens</td>
</tr>
<tr>
<td>$L$</td>
<td>distance between the two supports</td>
</tr>
<tr>
<td>$L_p$</td>
<td>bond length of the I-section</td>
</tr>
<tr>
<td>$M_u$</td>
<td>ultimate moment of beam specimens</td>
</tr>
<tr>
<td>$P$</td>
<td>applied load on the beam specimens</td>
</tr>
<tr>
<td>$P_1$</td>
<td>load at the end of the stage $S_1$</td>
</tr>
<tr>
<td>$P_2$</td>
<td>load at the end of the stage $S_2$</td>
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\( P_f \) failure load of beam specimens
\( P_p \) applied load at the loaded end in the push-out test
\( P_s \) ultimate bond load in push-out test
\( P_u \) ultimate load of beam specimens
\( P_{up} \) ultimate load in push-out test
\( P_y \) yield load of beam specimens
\( S \) average slope of load-midspan deflection curve
\( S_{1} \) the slope of the initial stage on the load-midspan deflection curve
\( S_{2} \) the slope of the initial stage on the load-midspan deflection curve
\( S_p \) relative slip between the I-section and the concrete at the loaded end
\( S_r \) residual slip in push-out test
\( S_s \) ultimate slip in push-out test
\( t \) total thickness of the FRP wrapped at the specimen
\( \Delta \) midspan deflection at ultimate load of beam specimens
\( \Delta_1 \) displacement of the loaded end in the push-out test
\( \Delta_2 \) displacement of the unloaded end in the push-out test
\( \varepsilon_a \) strain of a cross-section in the I-section
\( \varepsilon_b \) strain of a cross-section in the I-section
\( \sigma \) normal stress on the specimens in the direct shear test
\( \sigma_a \) stress of a cross-section in the I-section
List of notations

\( \sigma_t \)  
stress of a cross-section in the I-section

\( \delta_r \)  
confining axisymmetric radial pressure

\( \mu \)  
friction coefficient between the concrete and the I-section coupons

\( \mu_E \)  
ductility of beam specimens

\( \tau \)  
average bond stress

\( \tau_1 \)  
local bond stress between two adjacent cross-sections of the I-section

\( \tau_r \)  
residual bond stress in push-out test

\( \tau_s \)  
ultimate bond stress in push-out test

\( \tau' \)  
shear stress at the interface between the concrete and the I-section coupons
CHAPTER 1: INTRODUCTION

1.1 Preamble

This thesis presents a research study on the application of the Glass Fibre Reinforced Polymer (GFRP) I-section in the composite beams. The first phase of the study was the experimental study on the flexural behaviour of a new type of composite beams, which are reinforced with the GFRP I-section and longitudinal steel bars. It was then followed by a push-out test to investigate the bond behaviour of the GFRP I-section to concrete, and a bond stress-slip relationship showing a good agreement with the experimental results was proposed. Finally, a direct shear test was conducted to determine the friction coefficient between the GFRP I-section and the concrete.

1.2 Background

Fibre Reinforced Polymer (FRP) is increasingly adopted in civil engineering construction in the last two decades because of the excellent properties of corrosion resistance as well as the high strength-to-weight ratio. Extensive research studies have been conducted on using FRP to retrofit existing structures [1-4]. On the other hand, FRP composites (such as FRP bars and FRP pultruded profiles) can be exploited as a kind of standard construction product in new construction [5-8]. Due to convenient installation and tailorability (e.g. I-section, square tube or circular
tube), FRP pultruded profiles (FRP profiles) have gained significant research attention in recent years. All the I-section mentioned in this thesis refers to the GFRP I-section, unless otherwise specified.

The FRP profiles are suitable for the application as all FRP structures, such as building floor, cooling towers and offshore platforms [9-11]. Moreover, it can be used in combination with other materials to develop composite structures. Regarding composite structures, some tests were carried out to use the GFRP I-section to reinforce the beam specimens. Two kinds of typical composite beams are shown in Fig 1.1. The composite beam with Cross-section A (Fig 1.1a) is composed of a concrete block on the top and an I-section at the bottom [12]. In this case, the concrete is employed for compression and the I-section for tension. Another type of composite beam with Cross-section B (Fig 1.1b) was studied by encasing the I-section in the middle of cross-section [13]. Nevertheless, both FRP and concrete are weak in ductility due to the intrinsic material properties, thus causing a brittle failure and low load-carrying capacity of the composite beams with Cross-section A or Cross-section B. In general, the performance of the composite beams reinforced with the I-section should be improved by developing a more reasonable design.

In addition, the bond behaviour of the FRP profiles to concrete is also a major
research focus for composite structures. It has been confirmed that the performance of the composite structures traditionally depends on the properties of the concrete and reinforcement, as well as the bond behaviour between the two components [14]. Therefore, an adequate bond between the concrete and the reinforcement is very significant to ensure the performance of the composite structures. Nevertheless, the smooth surface of the FRP profiles causes a weak bond to concrete, thus further resulting in the poor performance of the composite structures [12, 13, 15-17]. Moreover, the effect of the bond behaviour of the FRP profiles on the structural performance is more apparent in comparison with steel bars or GFRP bars due to the larger surface of the FRP profiles. In order to achieve good composite actions for the FRP profiles, it is essential to understand the bond mechanisms between the concrete and the FRP profiles and investigate the bond-slip relationships.

Fig 1.1 Cross-sections of composite beams: (a) Cross-section A [12]; (b) Cross-section B [13]
Although the bond behaviour of the FRP pultruded profiles to concrete is significant for improving the performance of the composite structures, the research studies in this aspect are limited both in the test method and theoretical model. The existing investigation of bond-slip model of FRP can be divided into two series, FRP sheet/plates bonded to concrete [18-21] and FRP bars in concrete [14, 22, 23], while both series of bond-slip models are not suitable for the bond behaviour of the FRP profiles. For example, the bond-slip model for FRP sheet/plate bonded to concrete is not suitable for the FRP profile due to the different interface properties. Epoxy resin is traditionally used to provide the strong adhesion force for FRP sheet/plate, while no extra materials are employed to bond the GFRP profiles and concrete. Regarding GFRP bars, although no adhesive agent is used at the interface, the size effect cannot be ignored since the majority of FRP profiles have a much larger surface (i.e. GFRP I-section, GFRP tube) than FRP bars. Thus, it is important to investigate the bond behaviour of the FRP profiles to concrete.

Besides the investigation of the bond behaviour between the FRP profiles and concrete, the friction coefficient is also a significant parameter of the interface. Pull-out [23] or Push-out [24, 25] test is traditionally employed to investigate the bond stress-slip relationship of FRP (or steel) to the concrete. Nevertheless, the friction coefficient could not be determined by the pull-out or push-out test. Usually,
the friction coefficient is required in the finite element analysis to simulate the contact of two interfaces by using the theory of Coulomb friction [26]. Due to the lack of the investigation for the friction coefficient, the interface between the concrete and FRP pultruded profile is usually simplified as a rigid connection [6, 27]. However, this simplification is obviously not accurate due to the slip occurred between the concrete and FRP pultruded profile in the composite structures. Hence, it is essential to determine the accurate friction coefficient between two types of the material and fully investigate the interface properties.

![Cross-section of the proposed composite beam](image)

**Fig 1.2 Cross-section of the proposed composite beam**

Based on the above-mentioned discussion, this study presents an experimental study on the application of the GFRP profiles in the composite beams, and a new type of composite beam reinforced with the I-section and the tensile steel bars were
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proposed as shown in Fig 1.2. In addition, a push-out test and a direct shear test were then conducted to preliminarily investigate the bond behaviour of the interface between the GFRP profiles and the concrete.

1.3 Significance and objective

This new type of composite beams is an attempt to combine the advantages of the FRP profiles and steel bars to improve the flexural response of the composite beams. The FRP profiles are encased in the concrete to ensure sufficient flexural strength of the beam members, and the use of tensile steel bars aims to improve the ductility and the flexural stiffness. The fabrication of the proposed composite beams is traditional and convenient without any special construction procedure, such as drilling holes or welding, thus reducing the construction cost and labour force, which is significant for the practical implementation of this structure in civil engineering. Moreover, the preliminary study on the bond behaviour of the GFRP profile to concrete provides an important reference for using the GFRP profiles to reinforce the concrete structures.

The specific objectives of this research study in this thesis are presented as below:

(1) To obtain the flexural response of the proposed composite beams and estimate the performance of the encased I-section, analysis the effect of the location of
the I-section and the type of the longitudinal tensile bars on the flexural response;

(2) To investigate the bond behaviour of the I-section to concrete, assess the influence of using transverse stirrups and the sand coating on the bond behaviour;

(3) To propose a simple theoretical model of the bond stress-slip relationship for the I-section encased in the concrete;

(4) To determine the friction coefficient between the concrete and the pultruded profiles with the consideration of the effect of type and compressive strength of concrete.

1.4 Thesis outline

This thesis is composed of seven chapters including this introduction, and a brief summary of the remaining chapters is given below.

Chapter 2 presents an extensive literature review of the current research regarding the application of FRP profiles. The investigation about the material properties of FRP profiles is first described, including the experimental studies and finite element analysis, followed by a simple introduction related to the application of all FRP profiles structures. Then, the development of the composite structure reinforced with
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FRP profiles is summarised according to different structure members, namely column members and beam members. Finally, the methods to improve the ductility of the composite structures reinforced with FRP composites are discussed.

Chapter 3 summarises the test of materials properties for all the materials used in this study, including the steel bars, GFRP bars, concrete and the I-section. All the test method, the test setup and the corresponding standards are given in this chapter. The main parameters determined include the compressive strength of concrete, the tensile strength of steel bars and GFRP bars, the tensile strength and compressive strength of the I-section.

Chapter 4 describes the experimental results and the analysis of the flexural test, which is the main test in this research study. Five beam specimens were subjected to the four-point bending. The general observation of the beam specimens was presented, including the failure modes, the development of cracks, the relative slip between the concrete and the I-section. The analysis of the test results involved the ductility, the strain of the tensile reinforcements and the I-section, as well as the load-midspan deflection curves.

Chapter 5 presents a push-out test to investigate the bond behaviour between the
I-section and the concrete. Five specimens with different configurations were cast and tested. The variables included the bond length, longitudinal stirrups and sand coating. Then, the bond stress-slip curves and the distribution of the bond stress were given. Moreover, the theoretical analysis was conducted by proposing a constitutive relationship for the bond stress-slip relationship, and the predicted results were found to have a good agreement with the experimental results.

Chapter 6 gives the experimental results of the direct shear test, which is intended to determine the friction coefficient between the I-section and the concrete. The design of the specimens and the details of the test setup are presented in this chapter. The effect of the compressive strength and the type of the concrete on the friction coefficient was investigated. In addition, the relationship between the ultimate shear stress and the normal stress was studied by using a curving fitting, and then, the friction coefficient was determined by the slope of the fitting line.

Chapter 7 summarises the conclusions of the three tests in this thesis, and the recommendations for further research study are also highlighted.
CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

This chapter presents a literature review regarding the application of Fibre reinforced polymer (FRP) profiles in civil engineering, including the development of FRP profiles, the investigation of the material properties and the performance of the composite structures (e.g. column members and beam members) reinforced by FRP profiles. Moreover, the methods to improve the ductility of the composite structure and the bond behaviour of the interface between the concrete and the FRP profiles are also presented.

2.2 Types of FRP profiles

Pultrusion is a common technique to produce FRP profiles and the process of which is shown in Fig 2.1. This technique is efficient and is accomplished by pulling the raw fibres through a resin bath and then through a heated die [28], thus producing the FRP profiles with the same shape as the die. The resin used herein includes polyester or vinylester matrix [29]. The shape of the FRP profiles can be customed by adjusting the shape of the die as shown in Fig 2.2. The process is automated and continuous, and is beneficial for producing the FRP profiles with any length. Normally, the fibres are aligned along the longitudinal direction in the FRP profiles due to the limitation of the manufacturing process.
Based on the different types of the raw fibres, the common FRP profiles include two series, namely Glass Fibre Reinforced Polymer (GFRP) profiles and Carbon Fibre Reinforced Polymer (CFRP) profiles. Clearly, CFRP profiles are found to exhibit higher mechanical performance than the GFRP profiles due to the superior tensile properties, however, the higher cost of which limits the large-scale application in civil engineering. On the other hand, GFRP profiles have gained more attention in recent years due to the lower cost and sufficient structural performance. Therefore, this review mainly focuses on the research studies of GFRP profiles as well as the application of GFRP profiles in the composite structures.

Fig 2.1 Schematic image of pultrusion process [35]
2.3 Properties of FRP profiles

2.3.1 GFRP profiles

As already mentioned, GFRP profiles are being increasingly used as structural members in civil engineering application. Several experimental studies have been conducted to determine the material properties of the GFRP profiles. For example, Guades et al. [30] presented a systematic experimental study on determining the tensile strength and the compressive strength of GFRP square tubes. In this study, all the coupons for the material tests were extracted from the longitudinal direction of the tube since the majority of the fibres are laid out in this direction. The tensile test was conducted by using ISO 527 [31], and five coupons with a dimension of 12.7 mm × 38.1 mm were tested to determine the average tensile strength. The average
compressive strength was determined by using ASTM D695 [32], and the five coupons with a nominal dimension of 12.7 mm × 38.1 mm were tested. Besides the coupon test, a compressive test for the full-scale specimens was also conducted for comparison. By comparing the test results of the small coupons extracted from the GFRP tube and the full-scale specimens, it is confirmed that the value of the former is more accurate than the latter to reflect the material properties of the GFRP pultruded profiles.

The behaviour of GFRP profiles under different temperatures has been also investigated by several studies [33-36]. A representative experimental study was conducted by Aydin [36] to obtain the tensile and compressive strength of the GFRP profiles under 13 different temperatures (below and above 25°C). The test results show that the tensile strength reduced by 28% and the compressive strength decreased by 75% at 100°C in comparison with that at 25°C. When the temperature increased to 200°C, the GFRP profiles approximately lost 50% tensile strength and the compressive strength was completely lost. At the low temperature of -50°C, the loss of the tensile strength was 14% and the loss of the compressive strength was 5%. In general, GFRP profiles are very sensitive to the high temperature and relatively stable under the low temperature. Therefore, fire performance should be given more attention when GFRP profiles are employed in the high-temperature environment.
The long-term mechanical performance of the GFRP profiles was investigated in several studies [37-40]. For example, Bazli et al. [40] conducted a representative experimental study focusing on bending and compression of the GFRP profiles under the long-term harsh environment. This aggressive environment was a process of accelerated artificial ageing by using the sea water affected by different temperature, wetting and drying cycles, as well as alkaline solutions and acidic solutions. The specimen was immersed in the sea water for about five months and then tested. The experimental results show that the greatest degradation (by 41%) of the flexural strength and the compressive strength was revealed for the specimens immersed in the alkaline solution. For the specimens immersed in the acidic solution, the reduction of the flexural and compressive strength had reached to 31%.

Finite element analysis has been developed to numerically investigate the performance of the GFRP profiles [41-43]. The majority of the finite element models were developed within the framework of ABAQUS standard commercial software [26]. The GFRP profiles are usually considered to be anisotropy or orthotropic linear elastic material adopted in the model based on the available literature. Eight-node solid elements with reduced integration or four-node isoperimetric shell elements were employed to simulate the GFRP profiles. The elastic and strength properties were traditionally obtained by the standard material
test, however, some assumptions have to be introduced due to the difficulties to experimentally determine some mechanical parameters. The effect of both temperature and buckling can be estimated by the simulation.

2.3.2 Hybrid FRP profiles

The main advantages of GFRP profiles over the traditional building materials (such as steel) are the low self-weight, high strength and reduced maintenance requirements. However, some drawbacks of GFRP profiles cannot be neglected which have hindered the widespread use of GFRP profiles, for example, lack of ductility, high deformability and susceptibility to the buckling phenomenon. In order to address these issues, a methodology is proposed by combining CFRP and GFRP fibres together to improve the stiffness as well as their buckling behaviour.

Review of literature shows that two approaches had been adopted to integrate CFRP fibres into GFRP fibres to form a new type of the hybrid profiles. The first approach is to bond CFRP sheet onto the surface of the GFRP profiles (Fig 2.3a) by using the adhesive, such as epoxy resin. The other approach is to embed the CFRP mat into the GFRP profiles (in the flange or on the web) during the pultrusion process as shown in Fig 2.3b.
The hybrid FRP profiles in Type A can be fabricated manually in the lab without the special requirement for the equipment. Therefore, this design is widely employed to improve the performance of the FRP profiles. The review regarding this type of the hybrid FRP profile (Type A) can be found in Section 2.4.3. Regarding Type B, due to the complexity of manufacturing process, this type of hybrid FRP profiles only could be fabricated by the manufacturers in the factories rather than in the lab. Based on the advanced equipment and technology, the FRP profiles in Type B usually perform more stable mechanical properties in comparison with the FRP profiles in Type A.

Nunes et al. [44] investigated the structural behaviour of the FRP profiles in Type B experimentally and numerically. A total of six Series of beam specimens were tested by using four-point bending and the typical cross-sections are shown in Fig 2.4. The

![Fig 2.3 Typical hybrid FRP profiles: (a) Type A; (b) Type B](image)

Nunes et al. [44] investigated the structural behaviour of the FRP profiles in Type B experimentally and numerically. A total of six Series of beam specimens were tested by using four-point bending and the typical cross-sections are shown in Fig 2.4. The
specimens in Series S0 were bare GFRP I-section and this series was defined as the reference series. The specimens in other four Series (S1-S5) were hybrid profiles with different configurations. The CFRP mats were employed to reinforce the GFRP I-section both in the flanges and the web-flange junction. Fig 2.4 shows the specific configurations of the specimens.

The test results showed that all the hybrid profiles tested performed higher bending stiffness than the reference GFRP I-section, thus confirming the effectiveness of hybridization in improving the stiffness and the serviceability performance of the pultruded beam members. Nevertheless, the ultimate load of the hybrid profiles could not be improved significantly by introducing the CFRP mats, even some specimens presented lower flexural strength than the reference beam specimens. The reason of this may be that delamination occurred between CFRP mats and the GFRP profiles due to the different material properties, although so much effort had been made by the manufacturers to ensure a good bond between two types of materials.

Besides the flexural behaviour of the hybrid profiles, the compressive behaviour of the hybrid profiles was also studied by Nunes et al. [29, 45] experimentally and numerically. Similarly, the conclusions confirmed that the hybridization was effective in stiffening the structural members. However, the issue of lack of the
ductility and the sensitivity to the buckling for the GFRP profiles still could not be addressed by introducing the CFRP mats.

Fig 2.4 Reference GFRP and hybrid profiles experimental series: (a) S0; (b) S1; (c) S2; (d) S3; (e) S4; (f) S5 [29]

2.4 Use of FRP profiles

The FRP profiles can be employed in civil engineering application in the form of all FRP structures or the composite structures. All FRP structures refer to the structures
only composed of FRP composites. For composite structures, which are commonly
reinforced with FRP profiles and other building materials, such as concrete or steel
bars. This section reviews the use of FRP profiles in all FRP structures and
composite structures, including composite columns and composite beams.

2.4.1 All FRP structures

Regarding all FRP structures, so much effort has been made to explore the use of
this type of structures. In comparison with the common FRP composites (e.g. FRP
sheet and bars), FRP profiles have some specific advantages, such as ease of
installation, tailorability of the cross-section and higher flexural behaviour, which
are beneficial for developing a time-saving and efficient construction in civil
engineering. The most typical all FRP structure is FRP composite bridge, as such
this section reviews the development of the FRP composite bridges in the world as
well as the most significant issue for all FRP structures in practical application,
namely the connection of the FRP profiles.

2.4.1.1 FRP composite bridge

A bridge is the most typical all FRP structure and the first pedestrian FRP bridge in
the world was built in Israel in 1975 [46]. Since then, more FRP bridges have been
built in North American, Asia, and Europe. For example, the first cable-stayed,
GFRP deck and pylons bridge was erected at Aberfeldy, Scotland in 1992. This bridge joined two regions of the Aberfeldy golf course and crossed the river Tay. The self-weight of the structure was significantly reduced by using the GFRP profiles, as such no heavy machinery was used when assembling the bridges. The durability performance of this bridge was found to be satisfactory after 16 years [47].

Halgavor suspension bridge, which is one of the longest curved composite structures in Europe, has a 47 m span over the A30 road near Cornwall, England. This bridge was built in July 2001 and was designed for pedestrians, cyclists, and horses. All the components of the bridge were connected with bonded structural joints. The FRP deck had a 4 meter width and was manufactured by using resin infusion with vinyl-ester resin and an ultraviolet resistant gel-coat. The tailorability of the FRP profiles was demonstrated in the project, which is contributed to the easy installation during the construction process.

More constructions regarding FRP composite bridges can be found in other literature, for example, Hollaway [48] presented a review of FRP composites for civil infrastructure applications; Keller has developed a detailed review of all-composite bridges and buildings from 1997-2000 [9]. Based on this review, it is noted that the
use of all FRP structures is only limited within the small bridges for pedestrians. Regarding using all FRP structures in large bridges, there are still a lot of technical issues to be addressed.

2.4.1.2 Connection of FRP profiles

Although FRP profiles as a type of building material have the distinctive advantages such as high strength-to-weight ratio and tailorability, there are still some difficulties when using the FRP profiles in all FRP structures, for example, the design of FRP joints and connections. A reasonable design of the joints and connections is significant to ensure a good structural performance. Some design standards have been established for steel structures, thus guiding the setup of the bolts or welding points between the different components. Due to the similarity between the steel structures and FRP structures, some designs of FRP joints and connections are copied from steel design practice. However, the intrinsic characteristics of the FRP profiles are apparently different from steel structures, thus requiring a different theory to guide the design of FRP joints and connections.

The influence of geometry has been investigated in several studies [49-53], and the variables involved the ratio of width-to-hole diameter \((w/d_0)\), the ratio of end distance-to-hole diameter \((e_1/d_0)\) as well as plate thickness \((t)\). The experimental
results show that when the ratio of end distance-to-hole diameter ($e_1/d_0$) was more than 1.5, which had little effect on the failure strength. With the increment of the ratio of width-to-hole diameter ($w/d_0$), the corresponding ultimate connection resistance was increased. The coupons cut from different direction of FRP profiles present different ultimate connection resistance, and the larger resistance was found for the coupons extracted from the longitudinal direction.

The fastener parameters were also experimentally investigated [54], and the main parameter involved was the material of the fastener. The preliminary conclusions of this study show that the failure modes and the ultimate load were determined by the mechanical properties of the FRP profiles if the strength of the fastener was stronger than that of the FRP plate. If the fastener was weaker than the FRP plate, the failure modes and the ultimate load were governed by the fastener with little damage to the FRP plates. Similarly, the influence of the angle between applied tension and pultrusion direction, as well as the influence of lateral restraint and the joints with angles have been discussed [10, 55].

Based on the above-mentioned literature review, it could be found that considerable studies have been conducted regarding the connections and joints of FRP profiles. Nevertheless, there is currently no quantitative guidance for the design of
beam-to-column joints, and the mimicry of bare steel joint configurations is not totally appropriate for the FRP profiles joints due to the orthotropic properties of FRP. Therefore, more investigations are required regarding the connection of FRP profiles in future studies to implement the application of the FRP profiles in civil engineering.

2.4.2 Composite columns reinforced with FRP profiles

Besides the application as all FRP structures, FRP composite structures are recently becoming a major research focus, which is made of FRP profiles combined with other common building material (e.g. concrete and steel bars). This design aims to create innovative structural forms which are cost-effective and of high-performance. This section presents a detailed review regarding the development of the composite structures reinforced with GFRP profiles based on the different types of structures, namely column members and beam members.

First, in terms of the composite columns, extensive research has been conducted on FRP-confined concrete columns by using FRP jacket. The performance of this type of composite column is well understood in both the bond behaviour and the constitute model of confined concrete. However, the structural behaviour of columns reinforced with FRP profiles is only examined by a few limited studies. This is
because the material properties of FRP jacket and FRP profiles has an intrinsic difference. For FRP jacket, the majority of the fibres are wrapped in the hoop direction to provide the confinement for the concrete columns, nevertheless, the fibres in FRP profiles is mainly placed in the longitudinal direction thus causing a weak strength in the hoop direction. As a result, FRP profiles are traditionally not recommended to reinforce the concrete columns.

Hadi et al. [6] conducted an experimental study on the axial compressive behaviour of GFRP tube reinforced concrete columns. The GFRP tube is a type of FRP profiles manufactured by a pultrusion technology. The column specimens are fabricated by placing the GFRP tube into the concrete columns (Fig 2.5) to provide reinforcement in both the longitudinal and the transverse directions. The apparent advantages of

![Concrete](image1.png) ![GFRP tube](image2.png)

Fig 2.5 Composite columns reinforced with GFRP tube: (a) cross-section; (b) elevation [6]
this type of columns are the improvement of the fire performance for GFRP tube due to the tube surrounded by concrete. A finite element model was also developed to investigate the structural behaviour. The experimental results show that the existence of GFRP tube is effective in increasing the strength and the ductility capacity of concrete columns. In order to protect the concrete cover from premature spalling caused by fire or impact loading, several holes are drilled on the tube to integrate the concrete core and concrete cover. However, the strength and the ductility of the columns are affected to some extent since the fibres in the longitudinal direction were destroyed.

![Diagram of composite columns reinforced with GFRP I-section or C-section](image)

Fig 2.6 Composite columns reinforced with GFRP I-section or C-section: (a) GFRP I-section; (b) GFRP C-section [56]

The performance of the composite columns reinforced with the GFRP I-section or C-section was investigated by Hadi et al. [56]. The specimen is in the form of a
square concrete column with one I-section or two C-sections encased in the concrete as shown in Fig 2.6. The objective of the encased FRP profiles is to improve the load-carrying capacity of the column members. In addition, steel stirrups were employed to provide the confinement for the concrete. The advantage of this design for the FRP profiles is apparent that the buckling of the I-section or C-section can be prevented due to the surrounding concrete and confinement of stirrups. The experimental studies were conducted by using axial compressive test and eccentric compressive test with different eccentricities. The experimental results show that the specimens reinforced with GFRP profiles achieved higher ultimate load but lower ductility in comparison with the reference columns reinforced with steel bars as well as the composite columns reinforced with C-sections.

In general, the composite columns reinforced with GFRP profiles traditionally possess a high load-carrying capacity, nevertheless, the ductility of which is poor due to the intrinsic material properties of the GFRP profile. Since the column members traditionally resist the majority of load transferred from the beams and floor, the brittle failure mode will pose a threat to the safety of the whole building. Therefore, the composite columns reinforced with GFRP profiles are not recommended to be used in columns members, or should be employed in combination with steel materials to ensure the sufficient ductility.
2.4.3 Composite beams reinforced with FRP profiles

Due to the superior tensile performance, GFRP profiles are traditionally employed in the composite beams for tension and a considerable amount of studies have been conducted in this aspect. This section reviews the composite beams reinforced with FRP profiles based on the different cross-sections of the GFRP profiles, including hollow box section and I-section.

El-Hacha et al. [17] had proposed a new type of composite beam reinforced with GFRP hollow box section, CFRP sheet and a layer of Ultra-High-Performance-Concrete (UHPC). The design objective of this type of composite beam is to incorporate the high performance of each material together to achieve a higher structural performance based on its unique properties. Fig 2.7 shows the typical cross-section of this type of composite beam. The beam specimens with Type A cross-section consists of a GFRP hollow box section, a UHPC block cast at the top flange and a CFRP sheet bonded to the bottom flange. The connection between the GFRP hollow section and the UHPC is reinforced with a layer of epoxy adhesive and GFRP shear studs. Type B cross-section has a similar design with Type A cross-section, while the connection mechanism between the GFRP hollow box section and UHPC is different. The outer surface of the top flange of the GFRP hollow box section is bonded with a layer of coarse silica sand to improve the bond
resistance. Moreover, an additional UHPC block is cast inside of the GFRP hollow box, bonded at the top flange, aiming to enhance the anchorage for the GFRP shear studs. All the specimens were tested by using four-point bending.

![Diagram of composite beams reinforced with GFRP hollow box](image)

**Fig 2.7** Composite beams reinforced with GFRP hollow box: (a) Type A; (b) Type B

[17]

The test results show the addition of UHPC and CFRP sheet to the GFRP hollow box improves the flexural strength and flexural stiffness of the composite beams. The higher bond strength was provided when the GFRP hollow box was coated with the coarse silica sand compared with the interface only coated with the epoxy resin adhesive. Nevertheless, the performance of the composite beam was limited by the brittle material properties of the concrete and GFRP hollow box section, and the lack of the ductility was also revealed. In addition, the buckling of the GFRP hollow box section occurred at the middle span and the weakness at the flange-web joints was
detrimental to the flexural behaviour of the beam specimens. Similar research studies have been conducted by Chen et al. [57] and Chakrabortty et al. [58].

Another type of typical composite beam reinforced with GFRP hollow box section (i.e. GFRP tube) is proposed by Belzer et al. [59], which is fabricated by casting the concrete inside the box section. The unique advantage of this composite beam is that the GFRP hollow box section can be employed as the stay-in-place formwork and provides the flexural strength as well as flexural stiffness, finally reducing construction cost and time. The configurations of the specimens are shown in Fig 2.8. The effect of the bond between the tube and the concrete was investigated through the flexural tests of the specimens with different levels of adhesive between the
concrete core and the tube. Three types of specimens were cast, including concrete-filled GFRP tube, concrete-filled GFRP tube with epoxy coated at the interior surface of the flange, and concrete-filled GFRP tube with epoxy coated at all the interior surface. One empty GFRP tube without concrete inside was tested to be a reference beam.

The experimental results of Belzer’s test show that the concrete-filled tubes possessed higher flexural strength and stiffness in comparison with the empty GFRP tube. The composite action of the specimens was significantly affected by the level of adhesive between the concrete and the tube. The specimens, the GFRP tube of which were coated with more epoxy resin on the interior surface of the flange, showed better composite action and less relative slip. However, the brittle failure mode of the composite beams was observed during the test process, which may pose a potential threat to the safety of the structures. Moreover, the connection between this type of composite beams and the column members is a serious issue due to the limitation of the technology. Similar conclusions have been drawn by Ahmed [60]. Therefore, more investigation in future should focus on the beam-column joints and the ductility of this type of composite beam.

GFRP I-section (I-section) is another type of typical GFRP profiles used in the
Chapter 2: Literature review

composite beams and is commonly made of two flanges and one web with the same dimension or not. The I-section can be employed to reinforce the composite beams by being positioned on the tension side of the beam specimens or encased in the concrete. The former has been investigated widely by some researchers [12, 27, 61-64]. For example, Nordin et al. [12] conducted an experimental test regarding a type of composite beams reinforced with the I-section on the tension side, and the configurations of which is given in Fig 2.9. All the I-sections in this study were bonded with a layer of CFRP sheet at the bottom of the flanges, aiming to enhance the flexural strength and stiffness. The variable was the connection approach between the concrete block placed on the compression side and the top flange. As a reference specimen, no concrete was employed at this type of the specimens (Fig 2.9a). For the other two types of the beam specimens, the concrete block was connected with the top flange by using high strength shear connectors (Fig 2.9b) and epoxy resin (Fig 2.9c), respectively. All the specimens were subjected to a four-point bending load.

The test results of Nordin’s study show that the composite beams reinforced with concrete blocks at the compression side provided higher ultimate load compared with the specimens without the concrete blocks. The concrete was contributed to the improvement of the ultimate load. The composite action between the concrete and
the flange varied based on the different connection mechanisms. The composite beams, the concrete of which was bonded to the flange by using epoxy resin, showed higher flexural strength and stiffness than the composite beam using steel shear connectors. This investigation demonstrated that the I-section can be employed to develop a high-performance composite beam. However, some disadvantages for this type of beam cannot be neglected. For example, the stability of the composite beam was weak and the premature buckling may occur at the web; the I-section is vulnerable from the fire or high temperature without the protection of the concrete cover; the ductility should be improved due to the brittle material properties of concrete and the I-section.

Fig 2.9 Composite beams reinforced with GFRP I-section: (a) Type A; (b) Type B; (c) Type C [12]

In order to improve the stability of the composite beam reinforced with GFRP I-section, Kwan et al. [13] proposed a new type of composite beam which was
reinforced with the GFRP I-section encased in the concrete as shown in Fig 2.10a. The steel shear connections were installed at the both flanges of the I-section to reduce the relative slip between the GFRP I-section and the concrete (Fig 2.10b). Other parameters include the stirrups installed at the GFRP I-section (Fig 2.10c) and the barchip fibres mixed into the concrete (Fig 2.10d), aiming to improve the flexural behaviour of the composite beams. All the specimens were tested by using four-point bending.

Fig 2.10 Composite beams reinforced with GFRP I-section encased in concrete: (a) Type A; (b) Type B; (c) Type C; (d) Type D [13]

Due to the confinement from the surrounding concrete, the improvement of the stability of the encased I-section was apparent based on the experimental results of Kwan’s study. The composite beam reinforced with the I-section and stirrups showed a slightly higher ultimate load than other beam specimens. The highest
ductility was found in the specimen only reinforced with the shear connector, therefore, the composite beams reinforced with stirrups and barship fibre were not recommended in future practice due to the limited ductility.

2.5 Bond behaviour between the concrete and FRP profiles

The performance of the composite structure is dependent upon the properties of the concrete and the reinforcement, as well as the bond behaviour between the two components [14, 22]. In the case of reinforced concrete beams, the deformed bars with the coarse surface are employed to ensure sufficient bond behaviour. In the case of FRP reinforced concrete beams, the surface of the FRP bars is coated with sand or processed by using some other roughening measurements to improve the friction coefficient, aiming to ensure the transfer of the stress by bond from the bars to the concrete [23, 65]. For GFRP profiles, which have a larger and smoother surface than steel bars as well as FRP bars, therefore, it is essential to investigate the bond behaviour of the GFRP profiles to the concrete.

The pull-out test is the common experimental approach to investigate the bond behaviour for steel bars and GFRP bars to concrete [66-68], and some standards have been established to guide this test. However, the pull-out test cannot be adopted for the investigation of the bond behaviour for the GFRP profiles due to the
technical issues. The steel bars or FRP bars are traditionally convenient to be fixed at the test machine due to the small diameters, while fixing the GFRP profiles is different due to the irregular cross-sections of the profiles. In addition, GFRP profiles as a new type of material lack of corresponding standards to be the guides or references. As a result, the study regarding the bond behaviour of the GFRP profiles to concrete is limited so far.

Goyal et al. [69] conducted a typical test on the bond behaviour between FRP stay-in-place formwork and concrete. The FRP used in this study is a commercially available pultruded GFRP profiles plate with T-shaped ribs, and all the specimens were cast using the concrete with a compressive strength of 50 MPa. Before casting the concrete into the formwork, two types of bond treatments were employed, namely adhesive bonding and aggregate bonding. Adhesive bonding is developed by coating a thin layer of adhesive on the bottom of the formwork, and the concrete is cast after 10 minutes. In terms of aggregate bonding, the adhesive was also applied on the formworks initially and then sand grains were coated evenly on the wet adhesive. When the adhesive had been hardened totally after approximate two days, the concrete was poured into the formwork and cured for 28 days. The test was conducted by using a novel experimental setup to pull out the GFRP profiles.
The experimental results [69] show that the bond behaviour was poor for the specimens without the roughening treatment. For the specimens, the interfaces of which are roughened by using resin or aggregate, it is apparent that the application of the adhesive bonding can considerably improve the bond behaviour than using aggregate.

2.6 Improvement of ductility

The lack of the ductility of FRP composite, including FRP bar or FRP profiles, poses a serious threat to the serviceability performance of the composite structures reinforced with FRP composites. Therefore, much effort has been made to improve the ductility of the composite structures reinforced with FRP composites. The most typical approach is introducing a certain amount of steel materials to replace FRP composites in the composite structures.

Fig 2.11 Using steel bars to replace GFRP bars: (a) Type A; (b) Type B; (c) Type C

[70]
Lau et al. [70] investigated the flexural behaviour of a series of beams reinforced with hybrid bars, namely steel bars and GFRP bars, aiming to improve the ductility of the beam members reinforced with GFRP bars. The beam specimens herein comprised three series as shown in Fig 2.11: (a) beam specimens reinforced by steel bars, (b) beam specimens reinforced by GFRP bars, (c) beam specimens reinforced by both steel bars and GFRP bars. The GFRP bars were replaced by different numbers of steel bars to assess the influence of steel bars on the improvement of the ductility. All the specimens were tested by using four-point bending. The experimental results show that introducing steel bars to replace the GFRP bars is an effective approach to improve the ductility of the beam members. Moreover, the ductility of the beam specimens varied with the change of the numbers of the introduced steel bars.

![Steel I-section + GFRP bars](image)

Fig 2.12 Steel I-section + GFRP bars [71]
Besides introducing steel bars to improve the ductility, steel I-sections were also employed in the beam members to ensure adequate ductility as shown in Fig 2.12. A series of beam specimens reinforced with steel I-section and GFRP bars was tested by Li et al [71]. The parameters involved in this study include the ratio of the GFRP bars and the location of the steel I-section. The flexural behaviour of the beam specimens was investigated by using four-point bending. The experimental results of Li’s study show that these beam specimens possess good ductility due to the existence of the steel I-sections. In addition, the flexural strength and the stiffness of the specimens were also improved significantly due to the superior structural performance of the steel I-sections. The flexural strength of this type of composite beams could be predicted by the conventional beam theory.

Introducing the steel materials to ensure the ductility of the structural members can be adopted not only in beam members but also in columns members. The most typical configuration is FRP-concrete-steel double-skin tubular columns (DSTCs), as shown in Fig 2.13a, which are fabricated by using FRP composites outside of the column specimens to be the formworks and provide the confinement, and using steel tube (circular tubes or square tubes) inside of the column specimens to provide the ductility. The space between the FRP and steel is filled with concrete. Teng et al. [3, 72] have conducted a series of experiments to investigate the performance of these
columns, and proposed theoretical models to predict the structural performance. As an improvement of FRP-concrete-steel DSTCs columns, the steel tube inside of the columns could also be replaced by the steel I-section or other steel profiles as shown in Fig 2.13b, the experiments conducted by Yu et al. [73] confirmed the effectiveness of this design.

![Diagram of composite columns](image)

(a) FRP tube + steel tube; (b) FRP tube + steel I-section [73]

### 2.7 Numerical analysis of the composite structure

Due to the limitation of the measurement technologies, experimental studies cannot present the stress or strain distribution of the composite structures in detail. Thus, the numerical analysis method is developed to investigate the behaviour of the composite structures. The common commercial software for finite element analysis
in civil engineering includes ANSYS and ABAQUS. For simplification, the review only focuses on the finite element analysis carried out by ABAQUS. Based on the types of the materials used in the composite structures, this literature review regarding finite element analysis of the composite beams is developed from three aspects, concrete, GFRP profiles and the interface between concrete and GFRP profiles. Since the discussion about the finite element analysis of the GFRP profiles has been presented in Section 2.2, therefore, only the concrete and the interface between two components in the finite element analysis are reviewed as below.

2.7.1 Concrete

Three types of constitutive relationships of concrete were provided in ABAQUS, including concrete smeared cracking model, concrete brittle cracking model, and concrete damage plasticity model. Among three constitutive relationships, the concrete damage plasticity model is widely employed due to a general capability for modelling concrete in all types of structures. The inelastic behaviour of concrete is defined by using the concepts of isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity. The material parameters in this model include dilation angle, flow potential eccentricity, initial biaxial/uniaxial ratio, the ratio of 2nd stress invariant on the tensile meridian and viscosity parameter. Part of the parameters is default values in this model and the other parameters could be
found in some typical literature. For example, the dilation angle is defined as 31 by Nielsen and Hoang [74], and the compressive and tensile stress versus strain constitutive data can be referred to the studies conducted by Jankowiak and Lodygowski [75]. The accuracy of these data has been verified to be enough to simulate the concrete by several studies [76]. A three-dimensional finite element model is traditionally developed to simulate the concrete by using 8-node 3D linear brick elements (C3D8) [76-78].

2.7.2 Interface between the concrete and GFRP profiles

The surface-to-surface contact pair method is traditionally used for the contact between concrete and GFRP profiles [11]. A master-slave algorithm is used which means that a master surface and a slave surface should be selected in this model [26]. Then, the small-sliding formulation is suggested to be employed, which assumes that although two surfaces may have large motions, the relative sliding is little between two surfaces. Finally, a contact interaction property should be specified both in the normal and tangential directions. In the normal direction, the default contact is named as hard contact, indicating the normal pressure transferred in this interface without limitation. Once the normal pressure reduced to zero or negative value, two surfaces are assumed to be separated. For the tangential direction, the classical isotropic Coulomb friction model is used to calculate the shear stress, as
such a friction coefficient is required theoretically. The friction coefficient is usually assumed to be zero for the contact between concrete and GFRP profiles for two reasons: (a) the effect of the friction force is assumed to be neglected for the composite structures; (b) the friction coefficient at the interface is technically difficult to be measured. This assumption is relatively reasonable for the composite columns due to the weak shear stress at the interface. However, the effect of the shear stress in the composite beams is significant, since the tensile strength of the GFRP profiles should be transferred to the beam specimens by the shear stress at the interface.

2.8 Summary

This chapter presents a review of the development of FRP profiles in the aspects of the types and the mechanical properties. Furthermore, the application of FRP profiles in civil engineering is summarised from two aspects, all FRP structures and composite structures, respectively. The methodology to improve the bond strength and the ductility of the composite structures reinforced with FRP profiles are also reviewed.

Based on the literature review, it is clear that the use of FRP profiles in the composite structures have been investigated widely in recent years, thus providing a
significant reference for the implement of FRP profiles in civil engineering application. The lack of the ductility for the existing composite structures is still a serious issue, although some methods have been proposed to improve the ductility as already mentioned. For the FRP profiles to be widely accepted in practical, a new type of composite beam is proposed in this research study, aiming to promote the application of the FRP profiles. An experimental study was carried out to assess the flexural response of the proposed composite beams, including the flexural strength, flexural stiffness and ductility. In order to develop a good understanding on the bond behaviour of the FRP profiles to concrete, a push-out test and a direct shear test were then conducted.

Chapter 3 presents the determination of the material properties for all the materials used in this study, including steel bars, GFRP bars, concrete and GFRP I-section as well. The corresponding test setup and the specific operations are reported in this chapter.
CHAPTER 3: MATERIAL TEST

3.1 Introduction

The materials used in this study included concrete, steel bars, Glass Fibre Reinforced Polymer (GFRP) bars and GFRP I-section, and the determination of the material properties are presented in this chapter. The material properties tested herein comprised the compressive strength of the concrete, the tensile strength of the steel bars and GFRP bars, the compressive strength and the tensile strength of the GFRP I-section. The test methods, corresponding apparatus and the details of operations are also reported in this chapter.

3.2 Concrete

The concrete was ordered from a local supplier with 120 mm slump and a nominal compressive strength of 30 MPa. Table 3.1 shows the composition of the concrete. Cylinders with a diameter of 100 mm and a height of 200 mm were cast and tested to determine the compressive strength of the concrete by using AS 1012.9-1999 [79]. After demoulding the formwork, all cylinders were immersed into a curing tank filled with water until the testing day. The compressive strength of the concrete is 20.8 MPa in 7 days and 31.8 MPa in 28 days and the average compressive strength was obtained from three cylinders. Table 3.2 shows the test results for the compressive strength of the cylinders.
Table 3.1 Composition of concrete [80]

<table>
<thead>
<tr>
<th>Constituent (kg/m³)</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>285</td>
</tr>
<tr>
<td>Fly ash</td>
<td>100</td>
</tr>
<tr>
<td>Coarse aggregate</td>
<td>1135</td>
</tr>
<tr>
<td>Coarse sand</td>
<td>543</td>
</tr>
<tr>
<td>Fine sand</td>
<td>217</td>
</tr>
<tr>
<td>Water</td>
<td>170</td>
</tr>
</tbody>
</table>

Table 3.2 Compressive strength of concrete

<table>
<thead>
<tr>
<th>Sample</th>
<th>Age (days)</th>
<th>Diameter (mm)</th>
<th>Height (mm)</th>
<th>Compressive load (kN)</th>
<th>Compressive strength (MPa)</th>
<th>Average compressive strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7</td>
<td>100</td>
<td>204</td>
<td>160</td>
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</tr>
<tr>
<td>2</td>
<td>7</td>
<td>100.5</td>
<td>200</td>
<td>162</td>
<td>20.6</td>
<td>20.8</td>
</tr>
<tr>
<td>3</td>
<td>7</td>
<td>100</td>
<td>202</td>
<td>172</td>
<td>21.9</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>28</td>
<td>100</td>
<td>200</td>
<td>265</td>
<td>33.8</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>28</td>
<td>100</td>
<td>200</td>
<td>238</td>
<td>30.3</td>
<td>31.8</td>
</tr>
<tr>
<td>6</td>
<td>28</td>
<td>100</td>
<td>201</td>
<td>247</td>
<td>31.5</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>35⁺</td>
<td>102</td>
<td>200</td>
<td>270</td>
<td>34.4</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>35⁺</td>
<td>100</td>
<td>203</td>
<td>240</td>
<td>33.5</td>
<td>32.5</td>
</tr>
<tr>
<td>9</td>
<td>35⁺</td>
<td>101</td>
<td>201</td>
<td>232</td>
<td>29.6</td>
<td></td>
</tr>
</tbody>
</table>

*The specimens were tested at 35 days after the casting of concrete.
3.3 **Steel bars**

Two types of steel bars (N16 deformed bars and R10 plain bars) were employed in this research study. Tensile test on three samples of steel bars was conducted for each type of bars by using AS 1391(2007) [81]. The total length of the sample was 500 mm, and the extensometer gauge length and the gauge length were 340 mm and 80 mm, respectively.

The samples were tested by using the Instron testing machine located in the civil engineering laboratories at the University of Wollongong, Australia as shown in Fig 3.1.

![Fig 3.1 Tensile test of steel bars: (a) Tensile test of N16; (b) Tensile test of R10](image)
Table 3.3 Testing result of N16 and R10 deformed bars

<table>
<thead>
<tr>
<th>Bar</th>
<th>Properties</th>
<th>Sample 1</th>
<th>Sample 2</th>
<th>Sample 3</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Yield Load (kN)</td>
<td>118</td>
<td>117</td>
<td>117</td>
<td>117</td>
</tr>
<tr>
<td></td>
<td>Yield Strength (MPa)</td>
<td>585</td>
<td>584</td>
<td>584</td>
<td>584</td>
</tr>
<tr>
<td>N16</td>
<td>Ultimate Strength (MPa)</td>
<td>676</td>
<td>676</td>
<td>676</td>
<td>676</td>
</tr>
<tr>
<td></td>
<td>Yield Strain (%)</td>
<td>0.35</td>
<td>0.34</td>
<td>0.34</td>
<td>0.34</td>
</tr>
<tr>
<td></td>
<td>Elastic modulus (GPa)</td>
<td>199.5</td>
<td>198.6</td>
<td>199.5</td>
<td>199.2</td>
</tr>
<tr>
<td></td>
<td>Yield Load (kN)</td>
<td>23</td>
<td>24</td>
<td>24</td>
<td>31.4</td>
</tr>
<tr>
<td></td>
<td>Yield Strength (MPa)</td>
<td>301</td>
<td>305</td>
<td>310</td>
<td>309</td>
</tr>
<tr>
<td>R10</td>
<td>Ultimate Strength (MPa)</td>
<td>480</td>
<td>484</td>
<td>484</td>
<td>483</td>
</tr>
<tr>
<td></td>
<td>Yield Strain (%)</td>
<td>0.36</td>
<td>0.34</td>
<td>0.38</td>
<td>0.36</td>
</tr>
<tr>
<td></td>
<td>Elastic modulus (GPa)</td>
<td>195.7</td>
<td>185.4</td>
<td>196.4</td>
<td>192.5</td>
</tr>
</tbody>
</table>

* Determined by the 0.2% offset method.

Fig 3.2 Stress-strain relationship of steel bars: (a) Stress-strain curves of N16; (b) Stress-strain curves of R10
During the test, one axial extensometer was used to monitor the stress-strain response of the specimens. The samples yielded gradually and the rupture of which was observed in the middle of the samples in the final stage. The average value of the tensile strength of N16 bars was 584 MPa and it was 483 MPa for R10 bars. The elastic modulus was 199.2 GPa for N16 bars and 192.5 GPa for R10 bars. All the test results are listed in Table 3.3. The stress-strain relationship of two types of steel bars is given in Fig 3.2.

### 3.4 GFRP bars

The GFRP bars with a smooth surface and a nominal diameter of 12 mm were ordered from the Treadwell Group Company [82]. Due to the smooth surface of the GFRP bars, the nominal cross-section area was used for the stress calculation. Sand was manually coated onto the surface of the GFRP bars in the lab to enhance the bond strength between the surrounding concrete and the bars. The tensile test was conducted by following ASTM D7205 / D7205M [83], and the length of the sample was 1300 mm. Two steel tubes were employed as anchors and fixed by expansive cement at the two ends of GFRP bars as shown in Fig 3.3. The steel tube had a length of 400 mm, and the outer diameter and inner diameter were 40 mm and 30 mm, respectively. During the test, a layer of plastic wrap was wrapped onto the GFRP bars to eliminate the possible explosion of the fibres from the bars at the
failure load. Five samples of GFRP bars were tested, and the average tensile strength and the modulus of elasticity were 503 MPa and 25.6 GPa, respectively. The test was conducted by using the Instron testing machine located in the civil engineering laboratories at the University of Wollongong, Australia.

Fig 3.3 Tensile test of GFRP bars: (a) Schematic diagram of test; (b) Test setup

3.5 GFRP I-section

The I-section (200 mm × 100 mm × 10 mm/Height × Width × Thickness) used in
this study was manufactured by using pultrusion method and ordered from Treadwell Group Company [82]. The material test of the I-section included the determination of the compressive and tensile properties at both the flange and the web. Since the majority of the fibre is traditionally positioned in the longitudinal direction for the pultruded profiles, as a result, the material tests herein only focus on properties in the longitudinal direction. The coupons for the material test were extracted from the I-section at the web and flanges as shown in Fig 3.4 and Fig 3.5.

The compressive testing was conducted in accordance with ASTM D695 [32] and the nominal dimension of the coupon is 12.7 mm × 38.1 mm. The tensile strength was determined in accordance with ISO 527 [31] and the nominal dimension of the coupon is 25 mm × 250 mm. In total, 20 coupons were tested for the material test of the I-section. Ten coupons (five from the web and five from the flange) were tested to determine the compressive strength and the other ten (five from the web and five from the flange) to determine the tensile strength. A strain gauge was employed on each coupon to investigate the stress-strain relationship of the material. Fig 3.6 shows the test setup for the material properties of the I-section, and Fig 3.7 presents the typical stress-strain curves. All the test results are summarised in Table 3.4.
Fig 3.4 Dimensions of the coupon of the I-section (mm)

Fig 3.5 Coupons taken from the I-section
Fig 3.6 Test setup for material test of the I-section: (a) Tensile strength test; (b) Compressive strength test

Table 3.4 Tensile and compressive properties of the I-section

<table>
<thead>
<tr>
<th>Position</th>
<th>Dimensions of coupon (mm)</th>
<th>Properties</th>
<th>Averages and Sample Standard Deviations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flange</td>
<td>25 × 250</td>
<td>Tensile strength (MPa)</td>
<td>381.5 ± 8.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Tensile elastic modulus (GPa)</td>
<td>38.5 ± 4.2</td>
</tr>
<tr>
<td></td>
<td>12.7 × 37.1</td>
<td>Compressive strength (MPa)</td>
<td>214.2 ± 17.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Compressive elastic modulus (GPa)</td>
<td>26.9 ± 1.5</td>
</tr>
<tr>
<td>Web</td>
<td>25 × 250</td>
<td>Tensile strength (MPa)</td>
<td>353 ± 30</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Tensile elastic modulus (GPa)</td>
<td>32.88 ± 1.8</td>
</tr>
<tr>
<td></td>
<td>12.7 × 37.1</td>
<td>Compressive strength (MPa)</td>
<td>233.8 ± 18.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Compressive elastic modulus (GPa)</td>
<td>30.2 ± 8.5</td>
</tr>
</tbody>
</table>

Note: Tensile properties were determined based on ISO 527 (1997); Compressive properties were determined based on ASTM D695 (2002).
Fig 3.7 Stress-strain curve for material test of the I-section: (a) Typical tensile stress-strain curve; (b) Typical compressive stress-strain curve

3.6 Summary

The properties of the materials used in this study were determined in this chapter. In general, the compressive strength of the concrete and the tensile strength of the steel bars met the requirement in this study. However, the tensile strength of the GFRP bars was 503 MPa and the modulus of elastic was 25.6 GPa, which was only half of the strength of the common GFRP bars. Due to the low strength of the GFRP bars, the design of the composite beam specimens was adjusted. For GFRP I-section, the flange and the web showed similar tensile strength (and stiffness) and compressive
strength (and stiffness). The materials test provided a significant reference for the design of the specimens and the analysis of the experimental results in the following chapters.

In chapter 4, the flexural test of the composite beams reinforced with the I-section is presented. Based on the experimental results, the structural behaviour of the proposed composite beams is assessed.
CHAPTER 4: FLEXURAL TEST

4.1 Introduction

Flexural test is the main test in this research study aiming to investigate the flexural behaviour of the proposed composited beams. Five specimens with different configurations were tested by using four-point bending. This chapter presents the experimental program, test setup and instrumentations as well. In addition, the experimental results and the analysis are also presented in this chapter.

4.2 Specimen details

The configuration of this new type of composite beam is similar to that of the steel reinforced concrete beam as shown in Fig 4.1. The I-section encased in the concrete is mainly intended to improve the flexural strength and the corrosion resistance of the beam members. The tensile steel bars used in this composite beam aim to increase the initial flexural stiffness and the ductility of the composite beams. In fact, the concept of incorporating FRP and steel materials together to enhance the ductility has been proved to be effective by many researchers [70, 71, 84-86]. Steel stirrups are employed to confine the concrete and to enhance the shear strength of the beam members.
The advantages of this new form of composite beams are obvious when compared to the existing composite beams reinforced with the I-section, for example: (a) the fire performance is improved because the I-section is protected by the surrounding concrete; (b) the stability of the I-section is improved because it is encased in concrete; (c) the improvement of ductility due to the application of the steel bars; (d) better confinement of concrete due to the application of the stirrups, as a result, the bond strength between the concrete and the I-section can be improved; (e) improvement of the initial stiffness because of the higher stiffness of the tensile steel bars.

4.3 Experimental program

The materials involved in this test included the I-section, concrete and the steel/GFRP bars. The corresponding material properties have been determined in
Chapter 3. A total of five beam specimens were cast and tested in this experimental study, and the details of the specimens and the configuration of the cross-section are presented in Table 4.1 and Fig 4.2. All the specimens had an overall length of 2040 mm and a cross-section of 200 mm × 350 mm. The label of the specimens used in this study represents the type of the tensile bars and the location of the I-section. The first letter (S/F) in the label indicates the material of the longitudinal tensile bars used in the specimen, steel bars (S) or GFRP bars (F). The number followed is the reinforcement ratio of the beam specimens in percent, and the last letter M/B (middle/bottom) indicates the location of the I-section. For instance, Specimen S0.57B is the specimen reinforced by the steel reinforcement with the reinforcement ratio of 0.57%, and the I-section is positioned at the bottom of the beam specimen.

The specimens were divided into three groups, namely Reference group, Group S and Group F. The first group is a reference group, which includes a traditional reinforced concrete beam. This beam was reinforced with four tensile steel bars with 16 mm nominal diameter, and designed as an under-reinforced beam to ensure the failure of the specimens in flexure.

Group S contains the two proposed hybrid beams, namely Specimen S0.57M and Specimen S0.57B. Specimen S0.57M was reinforced with the I-section and two
tensile steel bars (Fig 4.2b), and the I-section was placed in the middle of the cross-section. Based on the previous studies, the location of the tensile materials could affect the flexural capacity of the beam members [71, 87, 88]. In order to investigate the effect of location of the I-section on the flexural strength, the I-section in Specimen S0.57B was transferred by 30 mm from the middle to the bottom of the cross-section. Apart from the different location of the I-section, the other configurations in Specimen S0.57B were identical with those in Specimen S0.57M, including the number of the transverse stirrups and the longitudinal bars.

Fig 4.2 Cross-sections of beam specimens (mm): (a) Beam RC; (b) Beam S0.57M; (c) Beam S0.57B; (d) Beam F0.46M; (e) Beam F0.46B
In order to investigate the influence of the steel bars on the ductility, the two tensile steel bars in Group S were replaced by three GFRP longitudinal bars with 12 mm diameter in Group F. The three GFRP longitudinal bars were expected to perform similar tensile strength as the tensile steel bars. As a result, all the tensile materials in Group F were GFRP materials. For example, Beam F0.46M was reinforced with the I-section and GFRP longitudinal bars as shown in Fig 4.2d, and the I-section was shifted down by 30 mm in Specimen F0.46B.

Steel transverse stirrups with hook angle 135° were used in each of the specimens. In order to facilitate the installation of the stirrups, two steel bars on the compressive side were employed as hangers for the stirrups. The steel stirrups and the steel bars on the compression side had plain 10 mm diameter with a nominal tensile strength of 250 MPa. The stirrups were spaced at 60 mm in the shear span and 80 mm in the pure bending region.
Table 4.1 Configuration of beam specimens

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen</th>
<th>Top bars</th>
<th>Bottom bars</th>
<th>Stirrups</th>
<th>GFRP</th>
<th>Location of I-section</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Material</td>
<td>Diameter (mm)</td>
<td>Number of bars</td>
<td>Material</td>
<td>Diameter (mm)</td>
</tr>
<tr>
<td>Reference</td>
<td>RC</td>
<td>Steel</td>
<td>10</td>
<td>2</td>
<td>Steel</td>
<td>16</td>
</tr>
<tr>
<td>Group S</td>
<td>S0.57M</td>
<td>Steel</td>
<td>10</td>
<td>2</td>
<td>Steel</td>
<td>16</td>
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<tr>
<td></td>
<td>S0.57B</td>
<td>Steel</td>
<td>10</td>
<td>2</td>
<td>Steel</td>
<td>16</td>
</tr>
<tr>
<td>Group F</td>
<td>F0.46M</td>
<td>Steel</td>
<td>10</td>
<td>2</td>
<td>GFRP</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>F0.46B</td>
<td>Steel</td>
<td>10</td>
<td>2</td>
<td>GFRP</td>
<td>12</td>
</tr>
</tbody>
</table>

aThe stirrups were spaced at 60 mm in the shear span and 80 mm in the pure bending region.
4.4 Preparation of specimens

Firstly, five steel cages were fabricated using thin steel wires to tie the stirrups and the longitudinal bars. Afterwards, in order to fix the I-section in the hybrid beams, the short steel wires were inserted into the flanges to eliminate any possible movement during the concrete casting as shown in Fig 4.3a. Two timber blocks were positioned under the I-section to adjust the location of the I-section in the middle or the bottom of the cross-section. Before moving the steel cages and the I-sections into the formwork as shown in Fig 4.3b, the plastic chairs were applied at the bottom of the steel cages to ensure a 20 mm cover. Due to the large size of the specimens, all the beam specimens were cured at ambient temperature. A wet hessian was employed to cover the specimens to prevent the moisture loss, and the specimens were watered during weekdays until the test day.

(a)                                 (b)
Fig 4.3 Fabrication of beam specimens: (a) Fixing GFRP I-section into steel cages; (b) Placing steel cages into formwork
4.5 Test setup and instrumentation

As shown in Fig 4.4, all the specimens were simply supported and subjected to four-point bending. Each of the beam specimens had a clear span of 1740 mm and shear span of 670 mm. The length of the pure bending region was 400 mm. For each of the specimens, five linear variable differential transformers (LVDTs) (1-5) were placed to monitor the deflection at different locations. Due to the possible brittle failure of the hybrid beams, the four LVDTs in the shear span (Fig 4.5a) were removed once the applied load reached 200 kN, which was about 50% of the expected ultimate load. The LVDT in the midspan was used to measure the deflection until the failure of the specimen. The wire rope of this LVDT was fixed at the bottom of the beam specimens, and the midspan deflection of the beam was measured according to the change of the length of the wire rope. A steel cover was made to protect this LVDT from the dropped concrete pieces (Fig 4.5b).

A series of strain gauges were affixed on the longitudinal bars and the I-section to investigate the strain distribution at the beam specimens. For Specimen RC, totally four strain gauges were attached to the longitudinal steel bars (Fig 4.6a). In terms of the hybrid beams, a total of ten strain gauges were installed at each specimen (Fig 4.6b). All the strain gauges were placed in the longitudinal direction.
Chapter 4: Flexural test

Fig 4.4 Test setup of beam specimens

Fig 4.5 Setup of LVDTs: (a) LVDTs in the shear span; (b) LVDT in the midspan
The displacement-controlled load was applied using the 1000 kN actuator. The loading rate was 1 mm per minute. After the peak load, once the load reduced to 80% of the ultimate load, the test of Specimen RC was stopped. For the hybrid beams, the specimens were considered to have failed once the tensile steel bars or GFRP longitudinal bars ruptured. All the test data were collected by a data logger connected to a computer.
Table 4.2 Experimental Results of Beam Specimens

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen</th>
<th>Yield load ($P_y$) (kN)</th>
<th>Ultimate load ($P_u$) (kN)</th>
<th>Midspan deflection at ultimate load ($\Delta$) (mm)</th>
<th>Failure mode</th>
<th>Ultimate Moment ($M_u$) (kN.m)</th>
<th>Ultimate Slip of I-section (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference</td>
<td>RC</td>
<td>380</td>
<td>380</td>
<td>12.2</td>
<td>Tensile steel bars yielded</td>
<td>127.3</td>
<td>-</td>
</tr>
<tr>
<td>Group S</td>
<td>S0.57M</td>
<td>313</td>
<td>413</td>
<td>36.6</td>
<td>Tensile steel bars ruptured</td>
<td>138.4</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>S0.57B</td>
<td>314</td>
<td>400</td>
<td>32.1</td>
<td>Tensile steel bars ruptured</td>
<td>134</td>
<td>9</td>
</tr>
<tr>
<td>Group F</td>
<td>F0.46M</td>
<td>-</td>
<td>357</td>
<td>22.9</td>
<td>GFRP bars ruptured</td>
<td>119.6</td>
<td>75</td>
</tr>
<tr>
<td></td>
<td>F0.46B</td>
<td>-</td>
<td>339</td>
<td>24.1</td>
<td>GFRP bars ruptured</td>
<td>113.6</td>
<td>80</td>
</tr>
</tbody>
</table>
4.6 Experimental results

The experimental results are summarised in Table 4.2. The yield load ($P_y$), ultimate load ($P_u$), failure mode and ultimate moment ($M_u$) have been presented. The yield load only can be obtained at Specimen RC and the composite beam specimens in Group S (S0.57M and S0.57B), since no yielding could be observed for the composite beam specimens in Group F (F0.46M and F0.46B) due to the brittle failure mode. Moreover, the bending stiffness, failure modes and crack propagation, as well as the relative slip between the I-section and the concrete are discussed in the sections below.

4.6.1 Load-midspan deflection curves

The load-midspan deflection curves are shown in Fig 4.7. Regarding the proposed composite beams in Group S, the ultimate load of Specimen S0.57M showed an 8% increase than that of Specimen RC, and the increase for Specimen S0.57B was about 5%. However, the specimens in Group F (Specimen F0.46M and Specimen F0.46B) performed lower ultimate loads than Specimen RC.

The two proposed composite beams (Specimen S0.57M and Specimen S0.57B) in Group S exhibited similar load-midspan deflection curves. The two curves had obvious yield points during the tests. For Group S, the stage before the yield points
(A) was named as Stage (O-A), and the curve between the yield point (A) and the ultimate point (B) was defined as Stage (A-B) (Fig 4.7). In Stage (O-A), the two curves had similar bending stiffness, and the loads of which increased up to about 300 kN where the both specimens yielded. Afterwards, the two curves increased in Stage (A-B) with similar slopes until the ultimate loads were reached. The ultimate load of Specimen S0.57M was 413 kN and it was 400 kN for Specimen S0.57B. After the ultimate loads, these two specimens failed and the loads started to decrease gradually. Finally, the curves of the two specimens experienced two sudden drops, which were caused by the rupture of the two tensile steel bars. The tests were terminated after the rupture of all the tensile steel bars.

Fig 4.7 Load-midspan deflection curves
In Group F, the load-midspan deflection curves of the two specimens also showed a similar trend. Initially, the two curves showed an almost linear increase and reached the ultimate load, 357 kN for Specimen F0.46M and 339 kN for Specimen F0.46B. Afterwards, the specimens failed and the load dropped suddenly, which was accompanied by continuous loud noise caused by the rupture of the GFRP bars. Finally, both beam specimens still could carry a stable but lower load. The tests of these two composite beam specimens were terminated due to the large slip that occurred between the I-section and the concrete.

4.6.2 Failure modes

The failure modes of all the specimens are clearly shown in Fig 4.8. All the specimens failed in flexure. Specimen RC (Fig 4.8a) is a traditional under-reinforced beam. As the applied load was increased, the tensile steel bars reached the yield strength and the specimen yielded. Afterwards, the concrete in the compression zone was crushed.

Regarding Specimen S0.57M (Fig 4.8b), several tiny cracks within the pure bending region were revealed in the initial stage of the test. The further increment of the load caused a prominent crack in the midspan, and then this crack propagated through the entire cross-section of the beam specimen. The concrete in the compression side
crushed. Finally, the two tensile steel bars ruptured with two big noises. Specimen S0.57B (Fig 4.8c) behaved in a similar failure mode to Specimen S0.57M, but the prominent crack developed more quickly and widely. Lastly, the tensile steel bars ruptured and the concrete crushed as well.

For specimens in Group F (F0.46M and F0.46B), which were reinforced with GFRP longitudinal bars and I-section, one prominent crack occurred below one loading point and then increased rapidly. Furthermore, the GFRP longitudinal bars ruptured suddenly at this crack with the increase of the applied load. The rupture of the GFRP longitudinal bars may be due to the stress concentration that occurred under the loading points. Finally, the beam specimen failed due to the rupture of GFRP longitudinal bars. The concrete in the compression side was still intact without failure when the test was terminated. No obvious cracks were observed within the shear span of the specimens in Group F.
Fig 4.8. Failure modes of beam specimens: (a) Failure mode of Specimen RC; (b) Failure mode of Specimen S0.57M; (c) Failure mode of Specimen S0.57B; (d) Failure mode of Specimen F0.46M; (e) Failure mode of Specimen F0.46B
In order to determine the accurate failure modes of the proposed composite beams, the strain-midspan deflection curves and the load-midspan deflection curve of the specimens were compared as shown in Fig 4.9. For specimen S0.57M, the strain of the top flanges (S6), the bottom flanges (S9) and the tensile steel bars (S10) were analysed to investigate the failure mode. At Point A in Fig 4.9a, it is clear that the tensile steel bars yielded due to the significant increase of the tensile strain, while the strain of the I-section increased steadily. The beam specimen yielded at the same time as the yield of the tensile steel bars, as such, the tensile steel bars governed the yield of the composite beams. Afterwards, the ultimate load was observed at Point B and the flanges of the I-section failed at the same time. Hence, the ultimate load of Specimen S0.57M was controlled by the I-section. A similar failure mode could be found for Specimen S0.57B as shown in Fig 4.9b.

Regarding the two specimens in Group F, the analysis of the failure mode is given in Fig 4.9c and Fig 4.9d. It is clear that the GFRP bars ruptured prior to the failure of the I-section, and the specimen failed with the rupture of the GFRP bars at the same time (Point C). Subsequently, the I-section failed due to the large deformation of the beam specimen, and the top flanges and the bottom flanges cannot reach the ultimate compressive or tensile strength. Therefore, the ultimate load of the composite beams in Group F is controlled by the GFRP bars rather than the I-sections.
Fig 4.9 Strain-midspan deflection curves versus load-midspan deflection curves: (a) Specimen S0.57M; (b) Specimen S0.57B; (c) Specimen F0.46M; (d) Specimen F0.46B
Based on the above discussion, the load-midspan deflection curves of the proposed composite beams were more clearly interpreted. In Stage (O-A) (Fig 4.7), the I-section and tensile steel bars resisted the load together, and then the tensile steel bars yielded at Point A thus leading to the yielding of the composite beam. In Stage (A-B), a further increase of the load was attributed to the superior flexural behaviour of the I-section. The failure of the I-section at Point B caused the failure of the composite beams, and the ultimate load was obtained at the same time. Afterwards, the specimens showed a very ductile response until the rupture of the tensile steel bars that occurred at Point C.

4.6.3 Bending stiffness

The bending stiffness ($EI$) of the beam specimens was compared based on the test results. The bending stiffness is calculated by:

$$EI = \frac{PL^2a}{48\Delta} \left(3 - \frac{4a^2}{L^2}\right)$$

where $P$ is the applied load on the beam specimens, $L$ is the distance between the two supports, $a$ is the distance from the support to the loading point and $\Delta$ is the midspan deflection. It should be noted that although two composite beams in Group S had different stiffness in Stage (O-A) and Stage (A-B), only the stiffness in Stage (O-A) was analysed in this calculation.
As shown in Table 4.3, the difference of the bending stiffness between Specimen RC and the two specimens in Group S was found to be minimal. Therefore, the composite beams reinforced with the I-section and tensile steel bars had similar bending stiffness compared with Specimen RC. It is believed that the high elastic modulus of the tensile steel bars contributed to the high bending stiffness of the beam specimens in Group S. The bending stiffness of both specimens in Group F was just 50% of that in Group S. The comparison of the bending stiffness between Group S and Group F indicated that the use of the tensile steel bars could ensure enough bending stiffness of the composite beams reinforced with the I-section.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Specimen S0.57M</th>
<th>Specimen S0.57B</th>
<th>Specimen F0.46M</th>
<th>Specimen F0.46B</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P$ (kN)</td>
<td>380</td>
<td>313</td>
<td>314</td>
<td>357</td>
</tr>
<tr>
<td>$L$ (mm)</td>
<td>1740</td>
<td>1740</td>
<td>1740</td>
<td>1740</td>
</tr>
<tr>
<td>$a$ (mm)</td>
<td>670</td>
<td>670</td>
<td>670</td>
<td>670</td>
</tr>
<tr>
<td>$\Delta$ (mm)</td>
<td>12.2</td>
<td>10.8</td>
<td>10.2</td>
<td>22.9</td>
</tr>
<tr>
<td>$EI \times 10^{12}$ (N.mm$^2$)</td>
<td>3.2</td>
<td>3.0</td>
<td>3.1</td>
<td>1.6</td>
</tr>
</tbody>
</table>
4.6.4 Ductility

The ductility definition used in this study ($\mu_E$) (Eq 4.2) is based on the energy theory which was proposed by Naaman and Jeong [89]. This equation could be used for calculating the ductility without identifying the yield point of the specimen, and it has been used in some previous studies [13, 90, 91].

$$\mu_E = \frac{1}{2} \left( \frac{E_T}{E_E} + 1 \right)$$

Eq 4.2

where $E_T$ is the total energy calculated based on the area under the load-midspan deflection curve. The $E_E$ is the elastic energy (Fig 4.10), which is computed by the area under the slope of the elastic behaviour. The $P_f$ in Fig 4.10 is the failure load of the specimen, where the tensile bars in the hybrid beams ruptured. Traditionally, the weighted value of $S_1$ and $S_2$ are employed to obtain the slope of elastic zone ($S$) below:

$$S = \frac{P_2S_1 + (P_2 - P_1)S_2}{P_2}$$

Eq 4.3

where $S_1$ and $S_2$ are the slopes of the initial two lines on the load-midspan deflection curve, $P_1$ and $P_2$ are the loads at the end of the two lines, respectively. The load-midspan deflection curves in Group F had no obvious two lines, so the ultimate load ($P_u$) was defined as $P_2$, and $P_1$ was equal to 0.5$P_u$ in this study.

Table 4.4 shows the ductility of the beam specimens. Fig 4.11 shows that the proposed hybrid beams in Group S performed higher ductility than the other beam
specimens. For example, the ductility of Specimen S0.57M was almost two times of that of Specimen RC. Nevertheless, the ductility of the specimens in Group F was really poor, and the ductility of both specimens was just 1.2. Therefore, it can be confirmed that the tensile steel bars can significantly improve the ductility of the hybrid beams reinforced with the I-section.

Fig 4.10 Ductility mode in this study

Table 4.4 Energy Ductility

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Slope $S_1$</th>
<th>Slope $S_2$</th>
<th>Slope $S$</th>
<th>Total energy $E_T$ (kN.mm)</th>
<th>Elastic energy $E_E$ (kN.mm)</th>
<th>Energy Ductility $\mu_E$</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC</td>
<td>36</td>
<td>0</td>
<td>36</td>
<td>20000</td>
<td>1800</td>
<td>6.1</td>
</tr>
<tr>
<td>S0.57M</td>
<td>35.2</td>
<td>3.5</td>
<td>27.5</td>
<td>29000</td>
<td>1136</td>
<td>13.2</td>
</tr>
<tr>
<td>S0.57B</td>
<td>34.6</td>
<td>3.8</td>
<td>28</td>
<td>22000</td>
<td>1395</td>
<td>8.4</td>
</tr>
<tr>
<td>F0.46M</td>
<td>24.1</td>
<td>11.4</td>
<td>17.8</td>
<td>5100</td>
<td>3592</td>
<td>1.2</td>
</tr>
<tr>
<td>F0.46B</td>
<td>23.6</td>
<td>9.6</td>
<td>16.6</td>
<td>5150</td>
<td>3474</td>
<td>1.2</td>
</tr>
</tbody>
</table>
4.6.5 Crack propagation

The distribution of cracks at ultimate load in the five specimens is given in Fig 4.12, and two different crack development modes were observed for the composite beam specimens. In Group S, the flexural cracks occurred in the pure bending region, and one prominent crack was found within the two loading points. However, in Group F, the prominent crack occurred under one loading point as shown in Fig 4.12d and Fig 4.12e.

In terms of the number of cracks, it was observed that the composite beams had fewer cracks than the traditional RC beam. Only a few flexural cracks occurred during the test within the pure bending region for the composite beams, and no shear
cracks were observed at the shear span. Since all the beam specimens had the same configurations of the stirrups, the disappearance of the shear cracks in the composite beams illustrated that the encased I-section could effectively improve the shear strength of the beam specimens.

Fig 4.12 Distribution of cracks at ultimate load: (a) Specimen RC; (b) Specimen S0.57M; (c) Specimen S0.57B; (d) Specimen F0.46M; (e) Specimen F0.46B

4.6.6 Slip between the I-section and the concrete

The slip between the concrete and the I-section is different for composite beams with different configurations. The relative slip between the I-section and the
concrete was measured by a steel ruler at the end of the test (Fig 4.13).

At the beginning of the test, no obvious difference of the slip was observed among the composite beams. However, after the ultimate load, two different slip modes were observed between Group S and Group F. In Group S, the slip slowly increased during the test, and the ultimate slip was about 10 mm as shown in Fig 4.13a and Fig 4.13b. For the specimens in Group F, the slip gradually increased before the ultimate load was reached. Afterwards, the slip showed a significant increase after the rupture of the GFRP bars until the termination of the test as shown in Fig 4.13c and Fig 4.13d. Based on the comparison between Group S and Group F, it is clear that the development of the slip between the I-section and the concrete was effectively controlled by using tensile steel bars in comparison with GFRP bars.

In general, it should be noted that the relative slip between the concrete and the I-beam occurred in all the composite beams, which is a detrimental effect on the improvement of the flexural strength of the composite beams. Studies should be conducted to investigate the bond properties of the interface, thus providing a reasonable reference for the application of the I-section in the composite beams. Therefore, a push-out test was conducted by the author and more details could be found in Chapter 5.
Fig 4.13 Slip of composite beams: (a) Specimen S0.57M; (b) Specimen S0.57B; (c) Specimen F0.46M; (d) Specimen F0.46B

4.7 Analysis and discussion

4.7.1 Steel bars

Based on the comparison between Group S and Group F, it is apparent that the proposed composite beams in Group S possess a very ductile response and high ultimate load. The tensile steel bars were significant for the proposed beams in
different stages of the test. First, due to the higher elastic modulus of the steel, the tensile steel bars showed a higher bending stiffness for the proposed composite beams in Stage (O-A). In Stage (A-B), due to the existence of the tensile steel bars, brittle failure could be avoided and the I-section contributed to further increase of the load. Therefore, the I-section could be more efficiently used in Group S than in Group F. Finally, the specimens failed at the ultimate load, while the tensile steel bars provided sufficient ductility to the beam specimens until the rupture of the tensile steel bars.

In general, the tensile steel bars could ensure the I-sections to be used more efficiently, while the brittle failure of the GFRP longitudinal bars limited the performance of the I-section. For example, the maximum tensile strain of the bottom flange in Specimen S0.57M was 0.00799 and in Specimen S0.57B was 0.00794, which was about 80% of the ultimate tensile strain (0.01) in the flange. Nevertheless, the maximum tensile strain of the bottom flange was no more than 70% of the ultimate strain in Group F, which was only 0.0068 in Specimen F0.46M and 0.0069 in Specimen F0.46B.
4.7.2 GFRP I-section

4.7.2.1 Flexural behaviour of the encased I-section

The I-section used in the composite beams provided both shear strength and flexural strength to the composite beams. Through the analysis of the crack propagation, few shear cracks in the composite beams confirmed the improvement of shear resistance offered by the I-section. In order to evaluate the flexural strength offered by the I-section, the tensile force provided by the bottom flange and the tensile bars before the ultimate load were compared in Fig 4.14. In fact, all the components of the I-section (the web, the top flange and the bottom flange) can provide flexural strength to the composite beams, for simplicity, only the tensile force offered by the bottom flanges were investigated in this study.

It is clear that the I-sections showed different performance before and after the yielding of the tensile steel bars as shown in Fig 4.14. Before the yielding of the steel bars, due to the large elastic modulus of the steel, the steel bars provided higher tensile strength than the I-section. The tensile force offered by the bottom flange was not more than 30% of the tensile force offered by the tensile steel bars in Stage (O-A). After the yielding of the steel bars, the stress of the steel bars did not increase and the I-section started to carry more load. Therefore, the tensile force of the flange increased significantly in Stage (A-B). When the ultimate load was reached, the
tensile force of the flange actually had exceeded the force of the steel bars as shown in Fig 4.14a. The large tensile force offered by the bottom flange confirmed the I-section could provide high flexural strength to the composite beam.

In Group F, the tensile force provided by the bottom flange increased significantly until the ultimate load was reached. The reason for this is that the I-section and GFRP bars have similar moduli of elasticity, so the increment of the stress for both components was similar. Due to the larger cross-section of the flange, the tensile force of the bottom flanges was larger than that of GFRP bars. For example, the cross-section of the bottom flange was about 3 times of the cross-section of the GFRP bars in Specimen F0.46M, as a result, the tensile force of the flange was always about 3 times of that in the GFRP bars.

Based on the comparison of the tensile force, it is believed that the I-section could offer high flexural strength to the beam specimens. Especially when the I-section and the tensile steel bars were used together, the two parts could carry the load at different stages of the tests. However, when the I-section and GFRP bars were used to reinforce the composite beams, the I-section failed quickly due to the brittle failure of the GFRP bars.
4.7.2.2 Effect of different flanges of I-section

The top flanges of the I-section were used for compression in the composite beams. Fig 4.15a shows the compressive strain curves of the top flanges in the composite beams. Initially, the stable increase of the compressive strain confirmed the top
flanges could offer the compressive strength to the beam specimens. After the ultimate load, the I-section failed and the compressive strain almost decreased to zero, which reflected that the top flange could not contribute to the flexural strength anymore.

The bottom flanges showed different behaviour in comparison with the top flanges as shown in Fig 4.15b. Initially, the bottom flanges could provide a large tensile strength, which was confirmed by the almost linear increase of the tensile strain. Afterwards, the maximum tensile strain and the ultimate load of the corresponding specimen were achieved at the same time. Finally, all the tensile strain was stable at a large value after experiencing a slight drop at the maximum strain. The large tensile strain after the ultimate load showed that the I-sections could still provide a high flexural strength even though the beam specimens had failed.

In addition, it is noticed that the maximum compressive strain was about 40%-70% of the ultimate compressive strain (0.0097) for the composite beams (Fig 4.15a). However, the maximum tensile strain reached about 70%-80% of the ultimate tensile strain (0.01) for all the composite beams (Fig 4.15b). Therefore, it is concluded that the bottom flanges can be more efficiently utilised when the I-section is encased in the concrete to reinforce the beam specimen.
Fig 4.15 Strain-midspan deflection curves: (a) Strain curves of the top flange; (b) Strain curves of the bottom flanges
4.7.2.3 Effect of locations of I-section

The I-section was positioned at two different locations in this study. Based on the test results, the ultimate load slightly decreased by 3% when the I-section was transferred by 30 mm from the middle to the bottom of the cross-section in Group S, and the decrease was about 5% in Group F. Since the decrease (3% and 5%) was small, the effect of the locations of the I-section was negligible in this study.

However, for the beam members, the load-carrying capacity should be improved when the tensile materials are placed closer to the tension side. As an initial assessment of the flexural behaviour of such composite beams, the randomness of the experimental results cannot be excluded in this study. More analyses should be conducted to investigate the effect of the locations of the encased I-section.

4.8 Summary

This chapter presents the test results of five beam specimens under four-point bending, including one traditional beam and four composite beams reinforced with GFRP I-section. The proposed composite beam in this study was reinforced with the I-section and longitudinal tensile steel bars. The parameters investigated include the location of the I-section and the type of the tensile bars. Based on the experimental results and analysis, the following conclusions are drawn:
1. The proposed composite beams possess a ductile response and higher ultimate load than the reference RC beam. The encased I-section can provide high flexural strength and additional shear strength, and the tensile steel bars can contribute to high ductility and ensure the bending stiffness of the composite beams.

2. The yielding point of the proposed composite beams is controlled by the tensile steel bars, and the ultimate load is governed by the I-section.

3. The bending stiffness and the energy ductility of the composite beams significantly decreased when the GFRP bars were used to replace the tensile steel bars.

4. The bottom flanges of the I-section are more efficiently utilised than the top flanges in the composite beams. Moreover, the bottom flanges can offer a high tensile strength even after the ultimate load, while the top flanges have almost negligible influence after the ultimate load is reached.

5. Slip occurs between the concrete and the I-section, which reduces the load-carrying capacity to some extent. Some roughening measures are suggested to improve the bond resistance at the interface, for example, sand coating or using additional mechanical connectors.

6. The locations of the I-section have little effect on the ultimate load of the beam specimens in this study. As an initial assessment of such composite beams, the
randomness of the experimental result should be taken into consideration, and more systematic studies are desirable to further evaluate the effect of different locations of the I-section.

This type of composite beam displays the superior flexural response in this preliminary evaluation, including the flexural stiffness, ductility as well as ultimate load. However, the relative slip could be observed in all the composite beams reinforced with the I-section. The slip is detrimental for the transfer of the bond stress between the concrete and the I-section, thus further affecting the improvement of the flexural strength of the beam specimens. Therefore, a push-out test was reported in Chapter 5 to investigate the bond behaviour between the concrete and the I-section.
CHAPTER 5: PUSH-OUT TEST

5.1 Introduction

As already mentioned in Chapter 4, the relative slip between the concrete and the I-section in the composite beams confirmed the weak bond strength at the interface. Traditionally, the performance of hybrid structures is dependent upon the properties of concrete and reinforcement, as well as the bond behaviour between the two components [14]. Therefore, an adequate bond between the concrete and the reinforcement is important for the performance improvement of hybrid structures [92]. The GFRP pultruded profiles usually have a larger surface when compared with GFRP bars or steel bars; hence, the influence of the bond behaviour on the structural performance is more significant. In order to achieve a good composite action for the GFRP pultruded profiles and concrete, it is essential to understand the bond mechanisms and determine the bond-slip constitutive laws.

The bond behaviour of GFRP pultruded profiles in concrete is investigated in this chapter. The GFRP pultruded profiles used was GFRP I-section. For the experimental method, the common pull-out test [67, 93] was not employed due to the difficulty of fixing the I-section in the testing machine, and a push-out test [24, 94] was adopted in this study. As a preliminary test, a total of five specimens with different configurations were tested. The parameters investigated included bond
length, transverse stirrups, and sand coating. Based on the experimental results, the failure modes, bond stress-slip curve and bond stress distribution are presented. Afterwards, the effect of stirrups and bond length is discussed, and the mechanism of the load transfer along the interface between the I-section and the concrete is analysed. Finally, a bond stress-slip constitutive model is proposed, and the predictions from this model are in close agreement with the experimental results.

5.2 Specimen details

The specimen for the push-out test is in a form of rectangular columns as shown in Fig 5.1, and the cross-section of which has the same dimension as that of the composite beam specimens in Chapter 4. The design of the specimens, including the dimensions of the cross-section as well as the space between the stirrups, referred to the dimensions of the composite beam specimens in Chapter 4. For all of the specimens, the I-sections of which were placed at the centre of the concrete, and the web was parallel to 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 wrapped on the surface of the I-section to debond the
concrete and the I-section within this region (debonding region). As shown in Fig 5.2, 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 [95]. This debonding region is beneficial for the push-out of GFRP I-section without the effect of the crush 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 for longitudinal reinforcement for ease of fabrication of the specimens.

![Fig 5.1 Specimen for push-out test](image-url)
5.3 Experimental program

A total of five specimens (Fig 5.4) were fabricated and tested, and Fig 5.3 shows two types of the cross-section for the five specimens, Section A-A (Specimens A and B) and Section B-B (Specimens AS, BS, and BSS). Both types of cross-section have a dimension of 200 mm width and 350 mm length. Table 5.1 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. Finally, 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 5.4a, 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 5.4b), and four longitudinal bars were used to fix the transverse stirrups. Specimen B (Fig 5.4c) was composed of the concrete and the I-section, and the bond length was 450 mm. The longitudinal bars and transverse stirrups were used in Specimen BS and Specimen BSS (Fig 5.4d). Moreover, the I-section in Specimen BSS was coated with sand in order to improve the friction at the interface.

Fig 5.3 Cross-section of specimens (mm) (See Fig 5.4 for elevation views): (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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Fig 5.4 Schematic diagram of specimens (mm) (See Fig 5.3 for sections A-A and B-B): (a) Specimen A; (b) Specimen AS; (c) Specimen B; (d) Specimen BS; (e) Specimen BSS
Table 5.1 Configuration of Specimens

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen</th>
<th>Cross-Section (mm)</th>
<th>Total height (mm)</th>
<th>Height of free end¹ (mm)</th>
<th>Bond length² (mm)</th>
<th>Height of debonding region (mm)</th>
<th>GFRP I-section (mm)</th>
<th>Stirrups (mm)</th>
<th>Longitudinal bars (mm)</th>
<th>Surface of the I-section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group A</td>
<td>A</td>
<td>350×200</td>
<td>400</td>
<td>50</td>
<td>300</td>
<td>50</td>
<td>200×100×10</td>
<td>-</td>
<td>-</td>
<td>Smooth</td>
</tr>
<tr>
<td></td>
<td>AS</td>
<td>350×200</td>
<td>400</td>
<td>50</td>
<td>300</td>
<td>50</td>
<td>200×100×10</td>
<td>Steel R10</td>
<td>Steel 4 R10</td>
<td>Smooth</td>
</tr>
<tr>
<td>Group B</td>
<td>B</td>
<td>350×200</td>
<td>600</td>
<td>100</td>
<td>450</td>
<td>50</td>
<td>200×100×10</td>
<td>-</td>
<td>-</td>
<td>Smooth</td>
</tr>
<tr>
<td></td>
<td>BS</td>
<td>350×200</td>
<td>600</td>
<td>100</td>
<td>450</td>
<td>50</td>
<td>200×100×10</td>
<td>Steel R10</td>
<td>Steel 4 R10</td>
<td>Smooth</td>
</tr>
<tr>
<td></td>
<td>BSS</td>
<td>350×200</td>
<td>600</td>
<td>100</td>
<td>450</td>
<td>50</td>
<td>200×100×10</td>
<td>Steel R10</td>
<td>Steel 4 R10</td>
<td>Sand coated</td>
</tr>
</tbody>
</table>

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 5.4 for the details)
5.4 Preparation of specimens

The preparation process of the specimens included cutting the I-section (Fig 5.5), attaching the strain gauges and casting concrete as well. Strain gauges were first attached in the longitudinal direction of the flanges and webs, and all the strain gauges were set up within the bond region as shown in Fig 5.6. A total of 10 strain gauges were attached at the I-section of Specimens A and AS, five strain gauges (S1 - S5) at the flanges and five (S6 – S10) at the web (Fig 5.6a). For Specimens B, BS and BSS, seven strain gauges (S11 – S17) were attached to the flange and further seven strain gauges (S18 – S24) were attached to the web (Fig 5.6b).
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Fig 5.6 Attaching strain gauges: (a) Strain gauges at Specimens A and AS; (b) Strain gauges at Specimens B, BS, BSS

Afterwards, the I-section attached with strain gauges 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 corresponding positions at 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 5.7a), and these two 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 transverse and longitudinal directions, respectively (Fig 5.7b).

Fig 5.7 Layout of I-section and steel cage: (a) Fixing I-section; (b) Fixing the steel cage

Concrete was ordered from a local supplier with a slump of 120 mm. The 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 demoulded (Fig 5.8) and cured in moist conditions until the test day.
5.5 Test setup and instrumentation

The push-out test was conducted using the 5000 kN testing machine. As shown in Fig 5.9, 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 to ensure adequate space for the push-out of the I-section. The displacement of the loaded end ($\Delta_1$) was 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 ($\Delta_2$), 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 this setup was 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.

Fig 5.9 Testing setup: (a) Schematic diagram of push-out testing; (b) Setup of test
5.6 Experimental results

The average bond stress ($\tau$) in this study is defined by:

$$\tau = \frac{P_p}{L_p C}$$  \hspace{1cm} (Eq 5.1)

where $P_p$ is the applied load at the loaded end, $L_p$ is the bond length of the I-section and $C$ is the perimeter of the I-section.

The slip ($S_p$) 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 ($S_p$) before pushing out the I-section is calculated taking into account the vertical extension of the specimen as below:

$$S_p = \Delta_1 - \Delta_2$$  \hspace{1cm} (Eq 5.2)

where $\Delta_1$ is the displacement of the loaded end and $\Delta_2$ is the displacement of the unloaded end.

After the I-section is pushed out, the displacement of the loaded end ($\Delta_1$) and the unloaded end ($\Delta_2$) kept the same increment, and the displacement of the unloaded end ($\Delta_2$) does not represent the vertical extension of the specimen anymore. Therefore, the slip ($S_p$) is equal to the displacement of the loaded end ($\Delta_1$):

$$S_p = \Delta_1$$  \hspace{1cm} (Eq 5.3)
5.7 Analysis and discussion

5.7.1 Failure Modes

Five specimens were cast and tested, and Fig 5.10 shows the failure modes of the specimens. 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 at 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 during the test, and which failed due to the premature compressive failure at the loaded end (Fig 5.11).
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Fig 5.10 Failure mode of specimens: (a) Specimen A; (b) Specimen AS; (c) Specimen B; (d) Specimen BS; (e) Specimen BSS

Fig 5.11 Compression failure of I-section
5.7.2 Bond Stress-slip Curves

The bond stress-slip curves of four specimens (A, AS, B, BS) are shown in Fig 5.12a, and the typical curve is shown in Fig 5.12b. In the first branch (O-A), the initial bond stress increased slowly. Afterwards, a roughly linear increase of the bond stress was revealed from Point A to the ultimate bond stress ($\tau_s$) 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 was reached. It should be noted that the ultimate bond stress ($\tau_s$) is obtained at Point B rather than Point D in this study, and the explanation is presented 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 ($\tau_r$) of the four specimens almost remained constant within 0.3-0.4 MPa. The experimental results of all the specimens are summarized in Table 5.2, including the ultimate bond stress ($\tau_s$), the residual bond stress ($\tau_r$) 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 5.12c, which shows 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 a limited experimental observation, so the interpretation of the bond stress-slip curves could not be developed in-depth only based on the experimental observation. More interpretation about these curves is given accompanied with the analysis of the strain of the I-section in the following parts.
Fig 5.12 Bond stress-slip curves at loaded end: (a) Bond stress-slip curves of Specimens A, AS, B, BS; (b) Typical bond stress-slip curve; (c) Bond stress-slip curve of Specimen BSS
### Table 5.2 Experimental results

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen</th>
<th>Ultimate bond load ($P_b$)</th>
<th>Ultimate load ($P_u$)</th>
<th>Ultimate bond stress ($\tau_u$)</th>
<th>Residual bond stress ($\tau_r$)</th>
<th>Ultimate slip ($S_u$)</th>
<th>Residual slip ($S_r$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group A</td>
<td>A</td>
<td>109.8</td>
<td>116.6</td>
<td>0.46</td>
<td>0.32</td>
<td>1.09</td>
<td>4.35</td>
</tr>
<tr>
<td></td>
<td>AS</td>
<td>72.1</td>
<td>99.7</td>
<td>0.30</td>
<td>0.29</td>
<td>1.04</td>
<td>4.45</td>
</tr>
<tr>
<td>Group B</td>
<td>B</td>
<td>184.6</td>
<td>193.5</td>
<td>0.51</td>
<td>0.36</td>
<td>1.61</td>
<td>5.02</td>
</tr>
<tr>
<td></td>
<td>BS</td>
<td>122.2</td>
<td>138.1</td>
<td>0.34</td>
<td>0.25</td>
<td>1.40</td>
<td>4.74</td>
</tr>
<tr>
<td></td>
<td>BSS</td>
<td>-</td>
<td>474.2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

1. Ultimate bond load is reached at Point B as shown in Fig 5.12b.
2. Ultimate load is reached at Point D as shown in Fig 5.12b.
5.7.3 Strain distribution of the I-section

The strain distribution extracted from the strain gauges at the flange and web of the I-section is shown in Fig 5.13 and Fig 5.14. Due to the similarity, the strain distribution of the I-section in Specimen A (Fig 5.13) is analysed as a typical strain distribution for Specimen A and Specimen AS, and the strain distribution of Specimen B (Fig 5.14) 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 5.13a and Fig 5.14a show the strain-load curves of the flange. 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 of the I-section. However, it was observed that the strain of S2 (or S12) is 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 cause the expansion of the web of the I-section, thus further resulting in a stress concentration occurred at the flange, in the position of strain gauges 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 given in Fig 5.13b and Fig 5.14b. The similar strain-load curves and the strain distribution were observed at the web for Specimen A (Fig 5.13c and Fig 5.13d) and Specimen B (Fig 5.14c and Fig 5.14d).

Fig 5.13 Analysis of strain (Specimen A): (a) Strain-load curves of flange; (b) Strain distribution along flange; (c) Strain-load curves of web; (d) Strain distribution along web
Fig 5.14. Analysis of strain (Specimen B): (a) Strain-load curves of flange; (b) Strain distribution along flange; (c) Strain-load curves of web; (d) Strain distribution along web.
The bond stress distribution along the I-section is also studied based on the strain difference between two strain gauges. As shown in Fig 5.15, the local bond force between two adjacent cross-sections could be calculated by:

$$\sigma_aA_s - \sigma_bA_s = \tau_1 dxC$$  \hspace{1cm} (Eq 5.4)

where $\sigma_a$ and $\sigma_b$ are the stress at two adjacent cross-sections; $A_s$ is the cross-sectional area of the I-section; $\tau_1$ is the local bond stress between two adjacent cross-sections; $dx$ is the length between two adjacent cross-sections.

The stresses $\sigma_a$ and $\sigma_b$ could be calculated by the corresponding elastic modulus ($E$) and the compressive strain ($\varepsilon_a$ and $\varepsilon_b$), so the local bond stress is calculated by:

$$\tau_1 = \frac{EA_s(\varepsilon_a - \varepsilon_b)}{dxC}$$  \hspace{1cm} (Eq 5.5)

It is noted that the elastic modulus ($E$) and the compressive strain ($\varepsilon_a$ and $\varepsilon_b$) 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 ($\tau_1$). As a result, the local bond stress ($\tau_1$) determined by Eq. 5.5 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. 5.1 is given in Fig 5.15b.
Fig 5.15. Local bond stress: (a) Calculation of local bond stress; (b) Comparison between local bond stress and average bond stress.
Fig 5.16. Typical bond stress distribution: (a) Bond stress distribution at flange (Specimen A); (b) Bond stress distribution at the web (Specimen A); (c) Bond stress distribution at flange (Specimen B); (d) Bond stress distribution at the web (Specimen B)
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The bond stress distribution at the flange and the web for Specimen A and Specimen B are shown in Fig 5.16. It is clear that the bond stress distribution is not uniform along the flange or the 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.

5.8 Discussion and analysis

5.8.1 Slip process

Based on the analysis of the strain and bond stress distribution as above-mentioned, the slip process of the I-section is analysed in Fig 5.17. The mechanics of stress transfer by the bond between FRP bars and concrete is mainly controlled by three factors [14, 96]: (a) chemical adhesion provided by the concrete; (b) friction due to the roughness of FRP bars; (c) mechanical interlocking offered by the deformation of FRP bars. The three mechanisms are not isolated during the slip process, and each mechanism has different performance in the different stages of the test. The mechanics of stress transfer by the bond between FRP bars and concrete can be used to analyse the bond behaviour of the I-section in concrete due to the similar material properties. The surface of the I-section is smooth, therefore, mechanical interlocking is ignored in this study, and only chemical adhesion and friction are 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 the bond region is intact without slip, and the bond force in the 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 5.17 indicate the different stages of the test, which have the same meaning as the letters in Fig 5.12b.

Fig 5.17 Slip process and bond stress distribution of I-section

(See Fig 5.12b for explanation of loading stages)
When the I-section was loaded in the initial stage (O-A), the interface between the concrete and the I-section was a bond region which provided the bond force to counteract the applied load. Afterwards, the different deformation between the concrete and the I-section 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 5.12b) 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), 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 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.
5.8.2 Effect of stirrups, bond length, and sand coating

Based on the analysis of the experimental results and the failure modes, it is clear that introducing stirrups did not improve the bond strength as expected. As shown in Fig 5.18, the ultimate bond stress ($\tau_u$) 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.

![Fig 5.18 Typical effect of stirrups on bond stress](image)

The influence of the bond length was investigated by comparing the specimens with different bond length (Fig 5.19). For specimens with the same bond length, the same
initial stiffness was observed even though stirrups were used in one of the specimens (Fig 5.18). For specimens with different bond length, the ultimate bond stress \( (\tau_s) \) of the specimen was improved by the longer bond length. For example, the ultimate bond stress \( (\tau_s) \) was increased from 0.46 MPa in Specimen A to 0.51 MPa in Specimen B due to the increase of the bond length.

![Graph showing typical effect of bond length on bond stress](image)

**Fig 5.19 Typical effect of bond length on bond stress**

The I-section in Specimen BSS was coated with sand to assess 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 \( (\tau_s) \) 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.

5.8.3 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 5.12), 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 are similar to the I-section, the bond stress-slip relationship of GFRP bars in concrete is referred to understand the bond behaviour of the I-section to concrete. Several bond stress-slip constitutive models for FRP bars have been reported and summarised in Table 5.3. Among these models, the model proposed by Eligehausen et al. [97] is the classical model. To start with, this model was applied to the bond of steel bars to concrete, then successfully used
for the bond behaviour of FRP bars to concrete by Rossetti et al. [98]. The bond stress-slip curve in this model is divided into different parts based on some representative parameters, such as the ultimate bond stress ($\tau_s$), the ultimate slip ($s_s$) and the empirical parameters $s_f$, $s_r$, $\alpha$ and $\beta$.

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

$$\tau = \tau_s \left( \frac{s_p}{s_s} \right)^{2.5} \quad (0 < s \leq s_s) \quad \text{Eq 5.6}$$

where $s_p$ is the slip at the loaded end and $\tau$ is the average bond stress. The experimental results of the ultimate bond stress ($\tau_s$) and ultimate slip ($s_s$) were used in this calculation. The comparison with a good agreement between the theoretical model and the experimental results are presented in Fig 5.20.

The proposed model in this paper requires the given ultimate bond stress ($\tau_s$) 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 ($\tau_s$) and the corresponding loaded end slip ($s_s$).
### Table 5.3 Existing bond–slip models for FRP bars

<table>
<thead>
<tr>
<th>Model</th>
<th>Ascending branch</th>
<th>Descending branch</th>
<th>Shapes of curves</th>
<th>Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Malvar model [99]</td>
<td>[ \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} ]</td>
<td>[ \tau_s = A + B \left[ 1 - \exp \left( -\frac{c_{\delta_o}}{f_t} \right) \right] ], ( s_s = D + E \delta_r )</td>
<td><img src="image1.png" alt="Graph" /></td>
<td>A, B, C, D, E, F, G = empirical constants determined for each bar type</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \delta_r ) = confining axisymmetric radial pressure</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( f_t ) = tensile concrete strength</td>
</tr>
<tr>
<td>Elieghausen et al. model</td>
<td>[ \tau = \tau_s \left( \frac{S}{S_s} \right)^{\alpha} ]</td>
<td>[ \tau = \tau_s - \left( \tau_s - \tau_f \right) \left( \frac{S - S_f}{S_r - S_f} \right) ]</td>
<td><img src="image2.png" alt="Graph" /></td>
<td>( \alpha, \beta ) = curve-fitting parameter</td>
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<td>(BPE model) [97]</td>
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<td>Shapes of curves</td>
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<td>BPE modified model [14]&lt;sup&gt;a&lt;/sup&gt;</td>
<td>$\tau = \tau_s \left( \frac{S}{S_s} \right)^{\alpha}$</td>
<td>$\tau = \tau_s \left[ 1 - p \left( \frac{S}{S_s} - 1 \right) \right]$</td>
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<td>$\tau_r = \beta \tau_s$</td>
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<td>$\tau = \tau_s \left[ 1 - \left( \frac{S}{S_s} - 1 \right)^2 \right]$</td>
<td>$\tau = \tau_s - \left( \tau_s - \tau_f \right) \left( \frac{S - S_f}{S_s - S_f} \right)$</td>
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<td>$\tau = \tau_s \left[ 1 - \exp \left( - \frac{S}{S_s} \right) \right]^\alpha$</td>
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<td>Tighiouart et al. [22]&lt;sup&gt;a&lt;/sup&gt;</td>
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<sup>a</sup>The values of $\tau_s$, $S_s$, $S_f$, $\tau_r$, $S_r$ was calibrated on the basis of the experimental results
Fig 5.20 Comparison of bond stress-slip curves: (a) Specimen A; (b) Specimen B; (c) Specimen AS; (d) Specimen BS
5.9 Summary

In this investigation, the experimental results and the bond stress-slip model on the 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 using a 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 ($\tau_u$) and the corresponding loaded end slip ($S_3$), 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 of concrete or the shape of the profiles, thus developing a more accurate bond stress-slip model. Moreover, it is noted that push-out test only can be adopted to investigate the bond-slip relationship, and the friction coefficient as a significant parameter of the interface cannot be determined by this test. Therefore, a direct shear test in Chapter 6 is reported aiming to experimentally determine the friction coefficient between the concrete and the pultruded profiles.
CHAPTER 6: DIRECT SHEAR TEST

6.1 Introduction

In Chapter 5, the pull-out test was shown to be an effective test method to investigate the bond stress-slip relationship of FRP pultruded profiles to the concrete, and a bond-slip model was proposed based on the experimental results. However, the friction coefficient as a significant parameter of the interface could not be determined by the push-out test. The friction coefficient is traditionally required in the finite element analysis to simulate the contact of two interfaces by using the theory of Coulomb friction [26]. Due to the lack of reference regarding friction coefficient, the contact between the concrete and the FRP pultruded profile is usually simplified as a rigid connection [6, 27]. Nevertheless, this simplification is apparently not accurate due to the relative slip that may occur between the concrete and the FRP pultruded profile in the hybrid beams, for example, the slip of the I-section in the composite beams mentioned in Chapter 4. As a result, it is essential to determine the friction coefficient between the two components to develop a more accurate finite element model.

In this chapter, the friction coefficient between the concrete and FRP pultruded profiles was determined by using the direct shear test method. The experimental results reported in this chapter include the failure modes of the specimen, shear
stress-displacement curves and the relationship between the ultimate shear stress and the normal stress. The friction coefficient between the GFRP profiles and concrete is determined within 0.4 – 0.5, and the adhesion stress is approximately 0.2 MPa.

### 6.2 Specimen details

The specimens in this study were composed of a concrete block (100 mm × 100 mm × 100 mm) and a coupon extracted from the I-section, and the two components were cast together when pouring the concrete. In order to investigate the influence of the location of the components on the friction coefficient, one coupon was cut from the flange and one from the web. As shown in Fig 6.1a, Coupon A was taken from the flange and had a T section with a flange width of 100 mm and web length of 50 mm. Coupon B was taken from the web and had the dimension of 100 × 100 mm. Due to the different shapes of the coupons, two types of specimens (Type A and Type B) were cast as shown in Fig 6.1b and Fig 6.1c. Type A refers to the concrete block with a coupon from the flange of the I-section. Type B refers to the concrete block with a coupon from the web of the I-section.
Fig 6.1 Two types of specimen (mm): (a) Two types of coupons; (b) Type A; (c) Type B

6.3 Experimental program

A total of 20 specimens were cast and tested, and the specimens were divided into five groups. The configurations of the specimens are shown in Table 6.1. In Group 30FS, the specimens were tested to determine the friction coefficient between the self-compacting concrete and the I-section, and the nominal compressive strength of the concrete in this Group was 30 MPa. The influence of the type of the concrete was investigated by using traditional concrete with a nominal compressive strength of 30 MPa in Group 30F.
For Group 40F, concrete with a nominal compressive strength of 40 MPa was employed to estimate the influence of the compressive strength of the concrete. The coupons in Group 40W were taken from the web of the I-section, aiming to assess the influence of the coupons from the different locations of the I-section on the friction coefficient. Group 40WN is a reference group, in which the specimens had the same configurations as the specimens in Group 40W, while the coupon and the concrete block for each specimen were artificially separated before the test. Therefore, the effect of chemical adhesion at the interface of the specimens in Group 40WN was eliminated during the test. One specimen in each group was tested under a nominal stress of 0.5 MPa, one at 1 MPa, one at 1.5 MPa and the final one at 2 MPa.

The label of the specimens consists of three parts. The first part is the number 30 or 40, which indicates the nominal compressive strength of the concrete, followed by a letter F or W indicating the coupons taken from the flange (F) or the web (W); this is then followed by a number to present the normal stress loaded in the specimen. In addition, the additional letter S in Group 30FS means that self-compacting concrete was used, and the letter N in Group 40WN indicates that the effect of the adhesion is eliminated.
### Table 6.1 Test matrix

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen</th>
<th>Compressive strength of Concrete (MPa)</th>
<th>Position of coupon</th>
<th>Normal pressure (MPa)</th>
<th>Type of specimens</th>
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The direct shear test requires the specimens to be tested under different normal stresses. Therefore, it is significant to determine the reasonable value of the normal stress. In order to reflect the accurate friction coefficient, the value of the normal stress should be close to the actual normal stress at the interface of the FRP pultruded profiles in the composite structures. However, it is traditionally difficult to achieve the normal stress loaded at the interface technically. In this study, the theory of FRP-confined concrete is referred to for determining the normal stress.

Over the last few years, a significant number of experimental and theoretical studies have been conducted on FRP-confined circular concrete specimens. The literature shows that when an FRP-confined circular concrete specimen is tested under axial compression, the concrete expands and causes the normal pressure on the FRP jackets as shown in Fig 6.2. The confinement pressure (i.e. the normal pressure) can be calculated by:

\[ f_l = \frac{2f_{FRP} t}{d} \]

where \( f_l \) is the lateral confining pressure, \( f_{FRP} \) is the tensile strength of the FRP in the hoop direction, \( t \) is the total thickness of the FRP wrapped at the specimen, and \( d \) is the diameter of the column specimens.
Table 6.2 shows a database collected from some previous experimental studies regarding FRP confining concrete columns. The ultimate confinement pressures in Table 6.2 were calculated by using Eq 6.1. The majority of the ultimate confinement pressures were found approximately to be between 5 MPa and 10 MPa. Normally, the actual confinement pressure is less than the ultimate pressure, and the larger normal pressure may cause the damage of the interface, affecting the accuracy of the test results. Therefore, the four normal pressures in this study were determined to be 0.5 MPa, 1 MPa, 1.5 MPa and 2 MPa. The test results have confirmed the rationality of the normal pressures determined by this method.

Fig 6.2 Confinement of FRP composite
## Table 6.2 Summary of the database

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<th>Height $H$ (mm)</th>
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<th>Depth of FRP $t_{frp}$ (mm)</th>
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6.4 Preparation of specimens

The first step of the fabrication of specimens was to cast the concrete block. The coupons were extracted from the flange and the web as shown in Fig 6.3a, and the interfaces of the coupons had a dimension of 100 mm × 100 mm. In order to cast the concrete and the coupons together, the coupons were fixed at one side of the formwork (Fig 6.3b), so the cross-section of the concrete block is same as the dimension of the coupons (100 mm × 100 mm). Afterwards, the concrete was manually cast and vibrated in the formwork. The specimens were demoulded after seven days and then cured in a moist environment until 28 days.

The shear test apparatus is originally designed for the direct shear test of the rock, and the dimension of the cross-section of the shear box is 120 mm × 120 mm, which is larger than that of the concrete block. In order to fix the specimen in the shear box, high strength plaster was filled into the gap between the concrete and the shear box. The concrete block was put into a custom plastic formwork which has the same dimensions as the shear box (Fig 6.4b). Next, the high strength plaster was poured into the formwork (Fig 6.4c). It is important to ensure the interface between the coupons and the concrete block to be level when casting the plaster. After 40 mins, the specimen encased by the plaster was demoulded and cured in the ambient environment. After seven days, the specimens were tested by using direct shear test.
Fig 6.3 Fabrication of specimens: (a) Coupon of I-section; (b) Formwork; (c) Casting concrete; (d) Specimens
Fig 6.4 Casting the plaster: (a) Specimen; (b) Specimen in formwork; (c) Casting the plaster; (d) Specimen with plaster
6.5 Test setup and instrumentation

The direct shear test is a laboratory or field test used by geotechnical engineers to measure the shear strength properties of soil or rock material. Also, several direct shear tests were conducted by using modified apparatus to obtain the friction coefficient between the steel and the asphalt [117] or between sand and steel [118]. Therefore, the direct shear test has been demonstrated to be an effective approach to determine the friction coefficient. The testing machine (Fig 6.5a) used in this study is the shear test apparatus at the University of Wollongong, Australia.

Fig 6.5b shows the test setup of this direct shear test for specimens in Type A. The specimens were fixed in the top shear box, and one steel plate was placed at the bottom of the bottom shear box to adjust the location of the shear interface, which was within the gap between the top shear box and bottom shear box. When the specimens in Type B were tested, the height of the steel plate was adjusted due to the different height of the specimen. In this case, a thin layer of kaolin was placed between the steel plate and the specimen to eliminate the effect of any space at the interface. When the setup was finished, the normal pressure was applied at the top shear box, and the shear force controlled by displacement was applied at the bottom shear box with a rate of 0.1 mm/min. The test was terminated when the ultimate shear load was reached.
Chapter 6: Direct-shear test

Fig 6.5 Test apparatus and setup: (a) Shear test apparatus; (b) Test setup
6.6 Experimental results

6.6.1 Failure mode

A total of 20 specimens were tested in this study. One specimen failed due to operation error, and the tests of the other 19 specimens were conducted successfully. All the specimens showed similar failure mode as shown in Fig 6.6a and Fig 6.6b. The coupons and the concrete block were separated totally at the interface after the peak load. The surface of the concrete was smooth and some slight marks were found on the surface of the coupons.

![Fig 6.6 Failure mode of specimen: (a) Concrete after shear test; (b) Coupon after shear test](image-url)
6.6.2 Shear stress – displacement curves

Fig 6.7 shows the shear stress-displacement curves of the five groups. Similar curves were revealed for Groups (30FS, 30F, 40F, and 40W). After some fluctuations in the initial stage, the shear stress experienced an almost linear increase before the failure of the specimen. The ultimate shear stress and the corresponding normal stress are summarised in Table 6.3. Although the specimens were tested under a constant normal load, it should be noted that the actual normal load was slightly fluctuated due to the effect of shear dilatancy. As such, Table 6.3 reports the nominal normal stress and the actual normal stress.

The shear stress-displacement curves of Group 40WN, which are shown in Fig 6.7(e), show different performance from the other four groups. For the specimens in Group 40WN, the shear stress linearly increased to the ultimate stress and then was kept constant after the failure of the specimens, and the sudden drop in the ultimate load in the other four groups could not be observed. Based on the difference of the shear stress-displacement curves between Group 40WN and Group 40W, it is confirmed that the sudden drop of shear stress in Group (30FS, 30F, 40F, and 40W) was caused by the loss of the chemical adhesion. Moreover, the ultimate shear stress of the specimens in Group 40WN was also slightly reduced compared with the specimens in Group 40W due to the loss of the chemical adhesion.
| Normal stress (MPa) | Group 30FS | | | Group 30F | | | Group 40F | | | Group 40W | | | Group 40WN |
|---------------------|------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
|                     | Actual normal stress (MPa) | Ultimate shear stress (MPa) | Actual normal stress (MPa) | Ultimate shear stress (MPa) | Actual normal stress (MPa) | Ultimate shear stress (MPa) | Actual normal stress (MPa) | Ultimate shear stress (MPa) | Actual normal stress (MPa) | Ultimate shear stress (MPa) |
| 0.5 MPa             | 0.53       | 0.50            | -                | -                | 0.57             | 0.51             | 0.5              | 0.53             | 0.52             | 0.34             |
| 1MPa                | 1.07       | 0.71            | 0.96             | 0.74             | 1.04             | 0.90             | 1.04             | 0.79             | 1.02             | 0.60             |
| 1.5 MPa             | 1.34       | 0.88            | 1.54             | 1.02             | 1.46             | 0.96             | 1.50             | 1.10             | 1.43             | 0.75             |
| 2 MPa               | 1.95       | 1.21            | 2.01             | 1.39             | 2                | 1.39             | 2.01             | 1.41             | 2.13             | 1.22             |

Table 6.3 Experimental results
Chapter 6: Direct-shear test

(a) 

(b) 

(c) 

(d)
6.6.3 Ultimate shear stress-normal stress curves

The ultimate shear stress and the corresponding normal stress were analysed by using curve fitting as shown in Fig 6.8. Based on the fitting results, the linear relationship between the ultimate shear stress and the normal stress is revealed and the relationship is shown as below:

\[ \tau' = \mu \sigma + c \]  

Eq 6.2

where \( \tau' \) is the shear stress at the interface, \( \mu \) is the friction coefficient, \( \sigma \) is the normal stress, and \( c \) is a constant. The fitting results illustrate that the shear stress \( (\tau') \) is composed of two parts, the friction stress \( (\mu \sigma) \) and a constant \( (c) \).
The influence of the type of concrete on the friction coefficient was investigated through the comparison between Group 30FS and Group 30F. The concrete in the two groups had similar compressive strength, but the concrete used in Group 30FS was self-compacting concrete and it was normal concrete in Group 30F. The friction coefficient in Group 30FS was 0.51, which is obviously smaller than that in Group 30F (0.62). This may be because the high deformability of self-compacting concrete caused a smoother interface of the concrete block, leading to a smaller friction coefficient.

The compressive strength of concrete had little effect on the friction coefficient based on the comparison of the friction coefficient between Group 30F (0.62) and Group 40F (0.58). To investigate the effect of the different components of the I-section, the concrete used in Group 40F and Group 40W had the same compressive strength. The coupons in Group 40F were cut from the flange and in Group 40W were cut from the web. No obvious difference was found for the friction coefficient between these two groups. Therefore, different components of the I-section have little effect on the friction coefficient.
Chapter 6: Direct-shear test

(a) $y = 0.58x + 0.21$
\[ R^2 = 0.944 \]

(b) $y = 0.62x + 0.16$
\[ R^2 = 0.9917 \]

(c) $y = 0.51x + 0.20$
\[ R^2 = 0.9907 \]

(d) $y = 0.60x + 0.20$
\[ R^2 = 0.9986 \]
Fig 6.8 Shear stress–normal stress relationships: (a) Group 30FS; (b) Group 30F; (c) Group 40F; (d) Group 40W; (e) Group 40WN

Table 6.4 Friction coefficient and adhesion stress

<table>
<thead>
<tr>
<th></th>
<th>Group 30FS</th>
<th>Group 30F</th>
<th>Group 40F</th>
<th>Group 40W</th>
<th>Group 40WN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction</td>
<td>0.51</td>
<td>0.62</td>
<td>0.58</td>
<td>0.60</td>
<td>0.54</td>
</tr>
<tr>
<td>coefficient</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Adhesion</td>
<td>0.20</td>
<td>0.16</td>
<td>0.21</td>
<td>0.20</td>
<td>0.04</td>
</tr>
<tr>
<td>stress (MPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Group 40WN was the reference group, in which the interface of the specimen had no adhesion stress. Compared with Group 40W, the variable (c) in Group 40WN was affected significantly and was almost reduced to zero (0.04 MPa), while the effect...
on the friction coefficient is negligible. Therefore, it can be concluded that the constant \(c\) in Eq 6.2 reflects the value of the adhesion stress.

Table 6.4 summarises the friction coefficient and the adhesion stress determined in this study. Clearly, the value of the friction coefficient is between 0.5-0.6, which was similar to the friction coefficient (0.57-0.7) between steel and concrete tested by Rabbat et al. [119]. However, the range of the adhesion stress (0.16-0.21) in this test was apparently smaller in the comparison with that (0.17-0.61) between the steel and concrete, which indicated that the interface between the steel and concrete show better bond behaviour than that between the GFRP pultruded profiles and concrete.

6.7 Summary

The friction coefficient between GFRP pultruded profiles and concrete was investigated in this study, and a total of 20 specimens were tested by using a direct shear test method. The parameters involved in this study included the type of concrete, the compressive strength of concrete and the different component of the I-section. Conclusions as below could be drawn based on the experimental results.

1. The direct shear test is an effective approach for testing the friction coefficient between the GFRP pultruded profiles and concrete, and the value of the friction
coefficient is between 0.5 and 0.6, and the adhesion stress is about 0.2 MPa.

2. Compared with the self-compacting concrete, the friction coefficient between the normal concrete and the GFRP profiles is larger.

3. The compressive strength of concrete and the different components of the I-section have little effect on the friction coefficient.

The friction coefficient tested in this experimental study is a significant parameter to develop an accurate finite element model for the hybrid structures reinforced with GFRP pultruded profiles. As a preliminary test, the parameters included in this study are limited, which may affect the accuracy of the experimental results. Clearly, more variables should be taken into the consideration in future studies to improve the accuracy of this friction coefficient.

In chapter 7, the general conclusions regarding this study and some recommendations are presented.
CHAPTER 7: CONCLUSIONS

7.1 Introduction

This thesis presents an experimental study on the flexural behaviour of a new type of composite beams reinforced with Glass Fiber Reinforced Polymer (GFRP) I-section and longitudinal steel bars. Since the apparent relative slip was observed between the I-section and concrete in the flexural test, a push-out test was then carried out to investigate the bond stress-slip relationship between the I-section and concrete. Finally, a direct shear test was conducted to determine the friction coefficient between the concrete and the I-section. The conclusions of the above-mentioned tests are presented below followed by some recommendations for future research studies.

7.2 Flexural test

Chapter 4 presents the experimental study on the flexural behaviour of the proposed composite beams. The parameters involved included the location of the encased I-section and the types of the tensile bars, and a traditional reinforced concrete (RC) beam specimen was tested as a reference beam. Based on the experimental results and discussion, some conclusions have been drawn as below:
1. The proposed composite beams possess a ductile response and higher ultimate load than the reference RC beam. The encased I-section can provide high flexural strength and additional shear strength; the tensile steel bars contribute to high ductility and ensure the sufficient bending stiffness of the composite beams.

2. The yielding point of the proposed composite beams is controlled by the tensile steel bars, and the ultimate load is governed by the I-section.

3. The bending stiffness and the energy ductility of the composite beams significantly decreased when the GFRP bars were used to replace the tensile steel bars.

4. The bottom flanges of the I-section are more efficiently utilised than the top flanges in the composite beams. Moreover, the bottom flanges can offer a high tensile strength even after the ultimate load. The influence of the top flanges is almost negligible after the ultimate load is reached.

5. Slip occurs between the concrete and the I-section, which reduces the load-carrying capacity of the beam specimens to some extent. Some roughening measures are suggested to improve the bond resistance at the interface, for example, sand coating or using additional mechanical connectors.

6. The locations of the I-section have little effect on the ultimate load of the beam specimens in this study. As an initial assessment of such composite beams, the randomness of the experimental result should be taken into consideration, and
more systematic studies are desirable to further evaluate the effect of different locations of the I-section.

7.3 Bond-slip test

The relative slip between the concrete and the I-section was revealed in the flexural test. To further investigate the bond-slip relationship between the I-section and concrete, a push-out test was conducted in Chapter 5 and five specimens with different configurations were cast and tested. The main parameters examined in this research study included the use of stirrups, bond length and sand coating. Some conclusions are drawn based on the experimental results and analysis:

1. The 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 using sand coating. Although the I-section with sand-coating could not be pushed out, the larger bond stress in 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 given ultimate bond stress ($\tau_s$) and the corresponding loaded end slip ($S_s$), therefore, a method for predicting these two parameters needs to be established.

7.4 Direct shear test

As a significant parameter of the interface, the friction coefficient could not be determined by the bond-slip test in Chapter 5. Therefore, a direct shear test was conducted to determine the friction coefficient between the concrete and the I-section in Chapter 6. A total of 20 specimens were cast and tested with different configurations. The parameters involved in this study were the compressive strength and the type of the concrete, as well as the coupons from different positions of the I-section (flange or web). The conclusions of the test are presented below:

1. A direct shear test is an effective approach to test the friction coefficient between
the GFRP pultruded profiles and concrete, and the value of the friction coefficient is between 0.5 and 0.6, and the adhesion stress is about 0.2 MPa.

2. Compared with the self-compacting concrete, the friction coefficient between the I-section and the normal concrete is larger.

3. The compressive strength of concrete and the different components of the I-section have little effect on the friction coefficient

7.5 Future research

The proposed composite beams in this study displayed superior flexural response both in the flexural strength and in the ductility. However, this experimental study is only a primary test to assess the flexural behaviour of the composite beams, as such the randomness of the experimental results cannot be eliminated due to the limitation of the data, and an accurate model to predict the flexural strength of the proposed composite beams is still not available. In order to develop a good understanding of the structural behaviour of the proposed composite beams, the following research studies in future should be conducted:

(a) Optimising the shape of the I-section needs to be carried out, since the top flange of the I-section could not be fully used and the wide top flange affects the performance of the concrete in the compressive region. The I-section with a
narrowed top flange will be tried to be used in the composite beams to improve the structural performance.

(b) Only the flexural behaviour of the proposed composite beams was investigated in this study, and the shear behaviour of which needs to be examined in the next step.

(c) Sand coating will be employed on the I-section to enhance the bond strength of the I-section to concrete, thus further improving the flexural strength of the composite beams.

(d) Besides four-point bending test, the composite beams need to be tested under more complicated loading conditions (e.g. cyclic load) to comprehensively assess the structural performance.
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