Bond Behavior between Basalt Fibres Reinforced Polymer Sheets and Steel Fibres Reinforced Concrete

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Abstract

Bonding behavior between FRP and Steel fibre reinforced concrete is not well studied. This study experimentally investigates the interfacial bond behavior between basalt fibre reinforced polymer sheet (BFRP) and steel fibre reinforced concrete (SFRC). Short steel fibres with four volume fractions were used to investigate the interfacial bond behavior of BFRP-SFRC as the mechanical properties of the concrete substrate (i.e. compressive strength and tensile strength) can be improved by adding steel fibres. The effects of volume fraction on the bond strength, effective bond length, local slip at the peak shear stress, and interfacial bond-slip relationship are evaluated and discussed. The experimental results showed that the debonding process becomes more ductile as the debonding plateau in the load and displacement curves has been significantly extended. Findings from the present tests show that the specimens with steel fibres of 0.25%, 0.50% and 1.0% experienced significant increase in the peak interfacial shear stress up to 31%, 53%, and 76% over the control specimen without steel fibres, respectively. In addition, the analytical bond strength models and interfacial bond-slip models, incorporating the effect of short steel fibres, are proposed.

Keywords: Basalt fibre reinforced polymer (BFRP); Steel fibre reinforced concrete (SFRC); Debonding.

1. Introduction

Normal concrete as the most common construction material is brittle and has very low strain capacity and resistance to tension and bending. Various short fibres such as natural, synthetic and steel fibres have been used to reinforce normal concrete. Since 1960s steel fibres reinforced concrete (SFRC) has been developed and used in civil engineering applications, especially for high-rise and long-span structures [1-3]. Adding steel fibres reinforced concrete (SFRC) has superior resistance to cracking and crack propagation due to the fact that steel fibres composites increase tensile strength at both pre-cracking and post-cracking stage [4]. As compared with normal concrete, SFRC is a ductile material as compared with normal concrete and the ductility can increase energy absorption capacity to resist seismic, impact and blast loads [5]. During service life of SFRC structures, structural strengthening is needed with the increasing requirement of load-carrying capacity [5-8]. Fibres reinforced polymer (FRP) is a popular strengthening material for reinforced concrete (RC) structures due to its low weight, high stiffness and strength to weight ratios [9]. Aramid FRP (AFRP), glass FRP (GFRP), and carbon FRP (CFRP) are the most common FRP composites. Recently, Basalt FRP (BFRP) has been widely used because it has not only excellent mechanical properties but also a competitive price [10]. For the FRP strengthened RC under different loading conditions, debonding is a dominant failure mode that is induced by a localized flexural or shear-flexural cracks formed in the concrete [11-15]. The debonding failure limits the strain capacity of FRP since it occurs at lower FRP strain than its ultimate strain [16]. Furthermore, the debonding failure cannot be prevented in FRP-strengthened RC structures due to the fact that the localized crack easily causes the debonding along the interface between FRP and plain concrete (PC) [17]. Therefore, adding fibres in RC members can improve structural performance by limiting microcracks and delaying FRP debonding.

In literature, a number of studies have investigated the flexural and shear behavior of FRP strengthened RC beams [12, 18-20] and FRP strengthened fibres reinforced concrete (FRC) structures [6-8, 17, 21, 22]. Benvenuti and Orlando [6] conducted finite element analysis on the FRP-strengthened SFRC beams and found that failure of beams was mainly dependent on the content of steel fibres. The addition of steel fibres significantly increased the load-carrying capacity and enhanced the ductility of the post-peak branch. Gribniak et al. [7] experimentally investigated the behaviour of steel fibres-reinforced concrete (SFRC) beams externally bonded with carbon fibres (CFRP) sheets under four points bending test. 20% increase of the ultimate deformation was observed from the CFRP-strengthened SFRC beams. Debonding of CFRP was also the premature failure and the failure process became gradual and visually apparent due to the added steel fibres. Li et al. [17] conducted an experimental study on hybrid FRP strengthened FRC beams. CFRP and GFRP sheets were used to improve the average rupture strain energy of the FRC beams. Short steel fibres with 0.9% and polymeric fibres with 0.1% volume fraction were used in the concrete mix. Three types of concrete beams were tested, which were plain concrete beams, polypropylene fibres (PF) reinforced concrete beams and PF/SF (steel fibres) hybrid reinforced concrete beams. The hybrid FRC beams strengthened with hybrid FRPs exhibited higher bending stiffness and the crack propagation can be controlled more effectively. Ibrahim et al. [21] investigated short-steel fibres (1.0% volume fraction) reinforced concrete (SFRC) beams retrofitted with GFRP laminates. The GFRP laminates retrofitted SFRC beams exhibited smaller crack spacing and the maximum increase of the ultimate load was found to be 130% as compared to GFRP laminates retrofitted RC beam. They also found that GFRP strengthened SFRC beams only exhibited FRP debonding induced by flexural cracks and none of the beams experienced delamination. In addition, the retrofitted SFRC beams exhibited higher ductility as compared to the retrofitted RC beams. Yin and Wu [8] conducted experimental and numerical studies on the structural performances of short SFRC beams with externally bonded FRP sheets. Four cases with different volume fractions of short steel fibres (0, 0.25%, 0.5%, and 1.0%) were mixed in the concrete beams. The concrete toughness was greatly improved by mixing short steel fibres. The failure mode of FRP-strengthened SFRC beams changed from the interfacial debonding to FRP rupture with a significant increase in the load bearing capacity. The numerical results found that the increase of the fibres volume led to the improved concrete toughness and the fracture energy.

The flexural capacity and shear capacity of FRP strengthened SFRC structures have been investigated [8, 17, 21]. The FRP-strengthened SFRC beams have similar shear stress and normal stress distributions to the FRP-strengthened RC beams. Interfacial shear stress reaches the peak value at the end part of FRP and decreases nonlinearly with the increasing distance away from the FRP end. The interfacial shear stress is much higher than the normal stress, which is resulted from stiffness change of the strengthened beam at end part of FRP sheet. In addition, it is found that the shear stress of FRP-strengthened SFRC beams with steel fibres is higher than that of the FRP strengthened RC beams and the effective FRP stress transfer length increases with the rising steel fibres volume fraction [8, 17]. It is worth noting that no study has been conducted to investigate the interfacial bond behaviour between FRP and SFRC. Therefore, it is important to investigate the interfacial bond behaviour of FRP-to-SFRC. For SFRC, the most important factors influencing the concrete mechanical properties are volume fraction (V_f) and aspect ratio (l/d) of fibres [23]. The volume fraction of steel fibres significantly influences the workability of concrete. The suitable volume fraction of fibres for concrete mixes ranges from 0.25% to 2.5% in volume fraction. The suitable aspect ratio of fibres for concrete mixes is between 50 and 100 [24]. To better understand the effect of short steel fibres on the bonding behavior between FRP and concrete, an experimental program was conducted by using the single-lap shear testing method, which is a reliable method as reported in the experimental program [25] and FEA program [26]. The digital image correlation (2D-DIC) techniques were used in this study to measure the full-field displacements and strains of the specimens. The bond-slip relationship between FRP and concrete can be experimentally obtained from the FRP strain distributions during the loading process. In addition, a fitting procedure was proposed and verified for obtaining the bond-slip curves. The effects of steel fibres on the interfacial bond strength, peak shear stress and the corresponding slip, effective bond length, and interfacial bond-slip relationship were also examined.

2. Experimental program

2.1 Material properties

Concrete prisms with a length of 350 mm, width of 150 mm and height of 150 mm were prepared as concrete substrates. Coarse aggregates with size of 5 - 10 mm was used in the test program. The short steel fibres with the length (L_f) of 25 mm and diameter (ϕ_f) of 0.30 mm (i.e. aspect ratio of 83.33) were used in the testing program, as shown in Figure 1. Four different volume fractions of short steel fibres, i.e. 0%, 0.25%, 0.5% and 1.0%, were used for the concrete with grade of 40 MPa. The Young's modulus, tensile strength, and density of the short steel fibres are 200 GPa, 2.5 GPa, and 7,800 kg/m^3, respectively.

The mechanical properties of PC and SFRC, including compressive strength and splitting tensile strength, were measured to investigate the correlations between the concrete material properties and the interfacial bond behavior. Three concrete cylinders with 100 x 200 mm from each batch of PC and SFRC were tested to obtain the axial compressive strength according to ASTM [27] and other three larger concrete cylinders of 150 x 300 mm dimension were tested based on ASTM [28] for the splitting tension tests after 28 days of curing. The obtained concrete material properties are summarized in Table 1.

Table 1 presents the details of the 12 tested specimens. The specimen ID was assigned to each specimen as "GX_PC/SFRCY_n_m". "GX" refers to the group from G1 to G4, and there are four groups in this study; "PC/SFRC" refers to the type of concrete, i.e. plain concrete (PC) and steel fibres reinforced concrete (SFRC); the letter "*n*" refers to the volume fraction of steel fibres; and the letter "*m*" means the specimen number from 1 to 3 (three identical specimens for each configuration).

A mixture of epoxy resin and hardener (West System 105 and 206) at a ratio of 5:1 was used to saturate the fibres. The epoxy resin has an ultimate tensile strength of 50.5 MPa, elastic modulus of 2.8 GPa and rupture strain of 4.5% [29]. Unidirectional basalt fibres (BFRP) sheets with a unit weight of 300 g/m^2 were used to bond the concrete substrates with two layers. FRP coupon tensile tests based on ASTM [30] were conducted to determine the mechanical properties of the BFRP sheets. The tensile strength, Young's modulus, rupture strain, and nominal thickness were 1,333 MPa, 73 GPa, 1.88%, and 0.12 mm, respectively.

2.2 Specimen preparations

The concrete prisms were demolded 24 hours after casting and then cured in water tank at Curtin University for 28 days. A total of 12 concrete prisms including 3 PCs and 9 SFRCs were prepared in this study. The details of the specimens and testing results are given in Table 2. All the specimens were cut in half in order to obtain the surface with more uniform distribution of steel fibres. The cutting surface of all specimens were roughened by a needle scaler to expose coarse aggregates. To prevent the effect of the protruding steel fibres on the bond capacity, any bulge was cut off. Manual lay-up procedure was conducted to bond the BFRP sheets onto the roughed surface of concrete substrates. The bond length of BFRP sheets was 200 mm for all the specimens in this study. A length of 50 mm was unbonded to eliminate the effect of concrete edge during the loading process, as shown in Figure 2.

2.3 Testing setup

The single-lap shear test was carried out by using the Shimadzu AGS-X 50KN series universal testing machine at Curtin University. All the specimens were tested in displacement control at a loading rate of 0.3 mm/min [31]. Figure 2 shows the testing setup for all the specimens. The machine has an inbuilt load cell to measure the load during the tests. Two strain gauges with 5mm gauge length were mounted onto the surface of BFRP sheets to measure the axial strain during the loading process. A camera together with digital image correlation (DIC) technique was also used to monitor the strain distribution of the BFRP sheets in all the tests. Three specimens were tested for each configuration to ensure the repeatability of the experimental results.

3. Experimental results and discussions

3.1 Failure mode

Failure mode reflects the performance and efficiency of the interfacial bonding [32]. Only one failure mode was observed in this study, i.e. debonding failure in the concrete prisms, where a thin layer of concrete with some steel fibres was attached to the BFRP sheets after debonding for the SFRC specimens. The debonding failure initiated at the loaded end for all the specimens, which was the same as a previous study [33]. The typical debonding failure photographs of the specimens after testing are shown in Figure 3 (a). Although the added short steel fibres had little effect on the failure mode, the debonding process between BFRP and SFRC was more ductile than the interface of BFRP-to-PC joints due to the pull-out behavior of fibres, which was shown in Figure 3 (b). With the increase of the volume fraction, more and more steel fibres were pulled out of the SFRC substrates.

3.2 Load-slip curve

As reported in the previous studies, there are three stages existing in the load-slip curves, which are the elastic stage, softening stage and debonding plateau stage [34]. After the elastic stage, microcracks develop in the concrete layer with the increasing shear slip. Figure 4 (a-d) shows the testing results of load and slip relation. The average debonding loads for the specimens

G1_PC, G2_SFRC_0.25, G3_SFRC_0.50, and G4_SFRC_1.0 were 8.50 kN, 9.23 kN, 10.03 kN, and 11.04 kN, respectively. The debonding loads increased with the volume fraction of steel fibres, which indicated that the bond strength between BFRP and concrete was improved by adding short steel fibres. In addition, the debonding process was more ductile as indicated by the longer softening stage and debonding plateau with the increasing fibres volume fraction. Also, four points (i.e. A, B, C, and D) were selected from the load-slip curves of the specimen G1_PC_1 in order to track the strain and the interfacial shear stress distributions at different loading stages.

There are fluctuations in load-slip curves due to the randomly distributed steel fibres and embedded aggregates in the adhesive-concrete layer. For the ease of comparison, a fitting procedure is conducted to smoothen the load and slip curves based on Equation (1) [25, 35-37]. This formula provides very accurate smoothening as the coefficient correlation R^2 for all the specimens is higher than 0.95. As given in Figure 5 (a), the coefficient correlation R^2 of the specimen G4_SFRC_1.0_2 is 0.96.

$$P = E_f t_f b_f \left(\frac{a}{b}\right)\left(1 - e^{-\frac{s}{a}}\right) \tag{1}$$

where P is the bond strength, E_f is the elastic modulus of FRP, t_f is the thickness of FRP, b_f is the width of FRP, a and b are coefficients, and s is the slip at the loaded end.

The fitted load-slip curves of BFRP-to-PC and BFRP-to-SFRC interfaces are plotted and compared in the same graph, as shown in Figure 5 (b). It can be seen that the interfacial bond strength increases with the steel fibres volume. This is because the tensile strength of concrete increases with the steel fibres volume. The splitting tensile strength of SFRC was higher than the PC by around 8-37%. For the specimens G4_SFRC_1.0, the specimen with 1.0% fibres volume had the highest split tensile strength, which is consistent with a previous study [38]. In addition, some existing interfacial bond strength models indicated that the bond strength between FRP and concrete correlates well with the tensile strength of concrete [34, 39]. In this study, the tensile strength is used to correlate with the interfacial bond behavior. The added steel fibres also tend to increase the toughness of the concrete (i.e. area under the stress-strain curve) [40], which results in higher fracture energy. In this study, pulling out of steel fibres from the concrete matrix can be observed when debonding occurs, as shown in Figure 3 (b). Higher force or energy is needed to debond and pull out the fibres from the concrete matrix. Also, the slope of the initial elastic stage of the load-slip curves increases with the volume fraction of steel fibres. This is because both the normal stiffness and tangential stiffness of the SFRC are improved by adding steel fibres [41].

In addition, the debonding plateau is significantly extended due to the increment of loaded end slip, as shown in Figure 5 (b). The testing results indicate that the ultimate slip at the loaded end of BFRP-SFRC was higher than the BFRP-PC by about 66-133%. This is because the softening stage of the concrete layer has been improved by adding steel fibres. The extension and propagation of microcracks in the softening stage were improved by the stress transferring capability of the added steel fibres. The improved softening of the BFRP-to-concrete interface can be observed in the load and slip curves and bond-slip curves, which will be discussed in the section of interfacial bond-slip relationship.

3.3 Interfacial shear stress and local slip calculation

Figure 6 shows the strain distributions along the loading direction at different loading stages. The fluctuations of the original strain curves extracted from the digital images can be observed due to the variations of the BFRP sheets [42] and the embedded aggregate in the adhesive-concrete layer [43]. Similar fluctuations derived from the DIC measurements in the interfaces of FRP-concrete, FRP-masonry, and FRP-steel were also overserved in the literature [44-46]. In this study, the strain distributions of all the specimens are similar and show a zigzag curve after debonding. Prior to the initial debonding, the strain at the loaded end increased with the applied load. After the initiation of debonding at the loaded end, a plateau was formed as the applied load increased until the final detachment of BFRP from the concrete substrates. It should be noted that the BFRP strain develops only within an active bonded region due to a certain range of shear stress concentration, which was the effective bond length.

To eliminate the fluctuations of the strain profiles, a non-linear curve fitting method was employed in this study, which can be expressed by Equation (2) [35]. It can be observed that the expression can reasonably simulate the strain distributions along the bonded length, as follows:

$$\varepsilon_{yy} = (\frac{a}{b}) / (1 + \mathrm{e}^{\frac{x - x_o}{b}}) \tag{2}$$

where ε_{yy} is the strain along the loading direction, a, b, and x_o are coefficients, and x is the distance from the loaded end. The ultimate strain for specimens G1_PC_1, G2_SFRC_0.25_2, G3_SFRC_0.50_1, and G4_SFRC_1.0_1 were 0.76%, 0.80%, 1.15%, and 1.37%, respectively, illustrating that the ultimate strain increased with the fibres volume. This is because the concrete toughness and crack resistance of concrete substrate were enhanced by adding steel fibres. The larger shear force results in larger deformation of BFRP sheets of the same stiffness, which is the reason why the specimens G4_SFRC_1.0 have the largest ultimate strain among the four groups. Overall, it can be seen that the strain distribution is sensitive to the steel fibres volume and the ultimate strain increased with the fibres volume.

The bond-slip relationship in the longitudinal direction can be obtained from the smoothened strain using Equation (3). The interfacial shear stress distribution within the bonded length can be evaluated by imposing the equilibrium condition of a FRP sheet with a length dx bonded to concrete, as follows:

$$\tau(x) = t_f E_f \frac{d\varepsilon_f}{dx} \tag{3}$$

where $\tau(x)$ is the shear stress at distance x from the free end of the specimen, $\frac{d\varepsilon_f}{dx}$ is the gradient of FRP strain along the bonded length, t_f is the FRP thickness, and E_f is the FRP elastic modulus. In addition, the local slip between FRP plate and concrete at distance x from the free end of the specimen can be calculated assuming a zero slip at the free end as follows:

$$s(x) = \int_0^\infty \varepsilon_f dx \tag{4}$$

The smoothened strain distributions measured from DIC can provide continuous strain, which has many more data points than the traditional method of using strain gauges. This is beneficial for the accuracy of the interfacial shear stress and local slip. Figure 6 shows the typical interfacial shear stress distributions along the BFRP sheets at different loading stages (A/B/C/D) for the specimens G1_PC_1 as indicated in Figure 4 (a). The interfacial shear stress distributions for all the specimens were similar due to the same shapes of shear stress distributions. After reaching the peak shear stress, the shear stress gradually decreased toward the free end with increasing the applied loads. The progressive debonding occurred when the shear stress was reduced to zero. The bell-shape shear stress curves shifted from the loaded end to the free end of the BFRP sheets along with the propagation of debonding. Theoretically, the peak shear stress should be a constant during the debonding process while the experimental peak shear stress decreased from the loaded end to the free end of the BFRP sheets. The possible reason is that the length of the interfacial shear stress transfer zone increased during the loading process, as evidenced by the interfacial shear stress distributions in Figure 6. It should be noted that the transfer zone of the interfacial shear stress can be defined from the interfacial shear stress distributions [47]

The results of the peak interfacial shear stress of all the specimens with different volumes are given in Table 2, which indicates that the steel fibres volume has significant influences on the peak shear stress due to the fact that the peak interfacial shear stress increases with the fibres volume. The average peak shear stress of the specimens G1_PC, G2_SFRC_0.25, G3_SFRC_0.50, and G4_SFRC_1.0 were 3.21 MPa, 4.21 MPa, 4.92 MPa, and 5.65 MPa, respectively. Compared to the control group G1_PC in terms of the peak interfacial shear stress, a growth of 31%, 53%, and 76% for the groups G2_SFRC_0.25, G3_SFRC_0.50, and G4_SFRC_1.0 can be obtained, respectively. In addition, the slip at the peak shear stress also increased with the fibres volume. The average slips of specimens G1_PC, G2_SFRC_0.25, G3_SFRC_0.50, and G4_SFRC_1.0 were 0.051 mm, 0.062 mm, 0.100 mm, and 0.122 mm, respectively. Based on the existing models of the interfacial shear stress of FRP-to-concrete [39], the interfacial shear stress is proportional to the tensile strength of concrete. As given in Table 1, the tensile strength of concrete increases with the of fibres volume because more fibres are active in arresting cracks [48]. Also, the measured strain increases with the fibres volumes. Because of the linear behavior of BFRP material, the shear stress developed in the BFRP sheets is proportional to the strain, indicating that the peak interfacial shear stress increases with the fibres volume. Meanwhile, the shear slip is also proportional to the strain as the shear slip is the integration of the strain, indicating that the shear slip also increases with the fibres volume. This means that the interfacial shear stress and the corresponding shear slip are sensitive to the added short steel fibres.

Figure 7 shows the experimental and predicted bond-slip curves. It can be observed that the bond-slip curves for the specimens with or without steel fibres exhibit the same trend: a non-linear ascending branch and a non-linear descending branch, which is consistent with the previous study [25]. The shear stress linearly increased to 40% of the maximum shear stress [42], after which it increased non-linearly and reached the peak shear stress. After the peak shear stress, a softening branch was observed where increasing the shear slip resulted in the reduction of shear stress. In addition, two existing bond-slip models proposed by Nakaba et al. [49] and Lu et al. [39] are used to compare with the experimental results, as shown in Figure

7. It can be observed that both the models cannot provide accurate predictions for the BFRP-to-SFRC interfaces due to the fact that the fibres-reinforcing index ($\frac{V_f L_f}{\phi_f}$) is not a parameter in the two bond-slip models. It is noted that adding fibre volume considerably increases the tensile strength of concrete but has very limited effect on the compressive strength. The compressive strength of concrete is considered in the model proposed by Nakaba et al. [49] while the tensile strength of concrete is considered in the model proposed by Lu et al. [39]. Therefore, the prediction on the peak shear stress by Nakaba et al. [49] increases less significantly with the increasing fibre volume as compared with the model proposed by Lu et al. [39]. In addition, adding steel fibres affects the interfacial fracture energy (i.e. the area under the bond-slip curve). The higher interfacial fracture energy the SFRC has, the higher shear stress of FRP sheets can be obtained, which is consistent with the findings by Yin and Wu [8]. As observed in Figure 7, both the models by Nakaba et al. [49] and Lu et al. [39] overestimate the peak shear stress of BFRP-SFRC. It is because the calibration factors used in the both models are for plain concrete not for SFRC. As a result, the calibration factors need to be adjusted for accurate predication of the peak shear stress at BFRP-SFRC interface. Thus, an accurate model needs to be proposed based on existing bond-slip models for FRP-to-SFRC joints.

As shown in Figure 7, the bond-slip relationship is sensitive to the added steel fibres as the peak interfacial shear stress and the corresponding shear slip increased with the fibres volume. Compared with the shear slip at the peak interfacial shear stress of the group G1_PC, a growth of 22%, 96%, and 139% for group G2_SFRC_0.25, G3_SFRC_0.50, and G4_SFRC_1.0 can be obtained, respectively. The increased local slip at the peak shear stress indicates that the elastic stage of the load and displacement relationship was improved by adding steel fibres. Also, the ultimate slip increased by the added steel fibres and the ultimate slip of specimens

G1_PC_3, G2_SFRC_0.25_2, G3_SFRC_0.50_1, and G4_SFRC_1.0_1 were 0.49 mm, 0.78 mm, 0.94 mm, and 1.21 mm, respectively. The extended ultimate slip indicates that the softening stage was improved by the added steel fibres. This is because the steel fibres improve the microcracking and crack propagation of the concrete substrates at both pre-cracking and post-cracking stages. SFRC is more ductile than PC and thus its energy absorption capacity was also improved, which is evidenced by the interfacial fracture energy of the interface between BFRP and SFRC. The experimental fracture energy obtained from Equation (5) are summarized in Table 2.

$$G_f = \int \tau ds \tag{5}$$

The interfacial fracture energy represented by the enclosed area of the bond-slip curve is summarized in Table 2. The added short steel fibres had a significant effect on the interfacial fracture energy. This is because as the fibres volume increases, higher fracture energy is needed to debond and pull out steel fibres from the concrete matrix. The specimens with higher fibres volume possessed a higher fracture energy due to the enhanced concrete properties. Compared to the specimens of FRP-to-PC joints, larger fracture areas can be observed for FRP-to-SFRC joints due to the improved concrete toughness and interlocking action, as shown in Figure 8 (a) and (b). The average values of the interfacial fracture energy for G1_PC, G2_SFRC_0.25, G3_SFRC_0.50, and G4_SFRC_1.0 were 0.452 N/mm, 0.568 N/mm, 1.275 N/mm, and 1.761 N/mm, respectively.

3.4 Effective bond length

The effective bond length (EBL) is the distance of the active bonding zone at ultimate debonding loads, which can be evaluated by using longitudinal strain fields of the BFRP sheets from the DIC analysis, as shown in Figure 9 (a). Successive digital images were analysed by using the DIC method, and the longitudinal strain field corresponding to each loading stage was derived [50, 51]. At the ultimate debonding load, the obtained strain distribution can be divided into three zones: (1) the fully debonded zone near the loaded end; (2) the stress transfer zone (the effective bond zone); and (3) the unstressed zone near the free end, as shown in Figure 9 (b). Dai et al. [35] proposed an equation to calculate the effective bond length, as:

$$L_e = 2bLn(\frac{1+\alpha}{1-\alpha}) \tag{6}$$

where L_e is the effective bond length, b is the coefficient obtained from Equation (2), and α is equal to 0.96.

Table 2 summarizes the effective bond length obtained from the DIC method and Equation (6). The average EBLs obtained from the DIC analysis as shown in Figure 9 (a) and (b) for specimens G1_PC, G2_SFRC_0.25, G3_SFRC_0.50, and G4_SFRC_1.0 were 68 mm, 66 mm, 57 mm, and 55 mm, respectively. The corresponding average EBLs for the four groups obtained according to Equation (6) were 73 mm, 63 mm, 59 mm, and 58 mm, respectively. There is a maximum of 7.27% difference on the results by using these two methods, which is an acceptable variation. Thus, both the methods can be used to obtain the EBL of BFRP-to-SFRC interface. The results show that the EBL decreases with the increase of steel fibres volume. This is because EBL is inversely proportional to the tensile strength of concrete [39]. Also, the strain distribution gradient of BFRP-to-SFRC interface is steeper than that of the BFRP-to-PC interface. This indicates that the EBL decreases with the increasing fibres volume as higher shear stress concentrates on the shorter stress transfer zone. In addition, similar patterns of strain contour development can be observed for all the specimens in Figure 9 (a). The shear strain contour in colours of red, yellow, and green refers to the shear stress distribution along the loading direction. A relatively uniform distribution of strain can be observed from the specimens G1_PC_1, however, the specimens G2_SFRC_0.25_2, G3_SFRC_0.5_1, and G4_SFRC_1.0_1 exhibited non-uniform strain fields. This larger variation of strain of the SFRC specimens can be attributed to the random distributions of steel fibres in the concrete substrates.

4. Theoretical verification and proposed models

4.1 Tensile strength of SFRC

For the specimens with the fibre aspect ratio of 83.3, the tensile strength of concrete increased with the volume fraction of fibres, as given in Table 1. Meanwhile, the added short steel fibres had minimal effects on the compressive strength of the concrete substrates in this study. The average compressive strength slightly decreased from 43.41 MPa (i.e. the volume fraction of 0.5%) to 43.13 MPa (i.e. the volume fraction of 1.0%).

Tensile strength of SFRC from splitting tensile tests is given in Table 1. It can be seen that the splitting tensile strength increases with the steel fibres volume, which is consistent with the previous study [52]. The increase of the fibres volume fraction from 0% to 1.0% results in an increase of 48.99% in the splitting tensile strength. Thus, the tensile strength of concrete will be correlated with the interfacial bond behavior in this study. Thomas and Ramaswamy [53] proposed a formula to predict the tensile strength of SFRC concrete as follows:

$$f_{SFRC} = A(f_{cu}^{'})^{\alpha 1} + B(f_{cu}^{'})^{\alpha 2}(RI) + C(RI) \tag{7}$$

where f_{SFRC} is the tensile strength of SFRC concrete, $f_{cu}^{'}$ is the standard cube compressive strength of the plain concrete and it is equal to 33.62 MPa. It should be noted that a factor of 1.2 is used to convert the cylinder strength to cube strength for plain concrete in this study [54]. In this study, *A*, *B*, and *C* are regression coefficients, both *α1* and *α2* are equal to 0.5, and RI is the fibres-reinforcing index ($\frac{V_f L_f}{\phi_f}$). After regression analysis, the coefficients (R^2=0.92) can be expressed as follows:

$$f_t = 0.628(f_{cu}^{'})^{0.5} + 0.165(f_{cu}^{'})^{0.5}\left(\frac{V_f L_f}{\phi_f}\right) + 1.404\left(\frac{V_f L_f}{\phi_f}\right) \tag{8}$$

4.2 Interfacial shear strength

A number of analytical models of bond strength have been proposed to predict the ultimate debonding strength between FRP and concrete. The analytical models by Lu et al. [39] and Dai et al. [55] are adopted in this study to predict the ultimate bonding strength as the both models considered interfacial fracture energy, which increased with the rising volume fraction of the steel fibres, as shown in Table 2. The specimens with steel fibres of 0.25%, 0.50% and 1.0% experienced significant increase in the interfacial fracture energy up to 20%, 64%, and 74% over the control specimen without steel fibres, respectively. As compared to the model by Dai et al. [55], the model by Lu et al. [39] considering the tensile strength of concrete yields more accurate results in this study since the tensile strength of concrete instead of the compressive strength is the main factor determining the bond strength.

The model proposed by Lu et al. [39] is presented as follows:

$$P_{max} = \beta_l b_f \sqrt{2 E_f t_f G_f} \tag{9}$$

$$G_f = 0.308 \beta_w^{\,2} \sqrt{f_t} \tag{10}$$

$$\beta_w = \sqrt{\frac{2 - b_f / b_c}{1 + b_f / b_c}} \tag{11}$$

The model proposed by Dai et al. [55] is presented as follows:

$$P_{max} = (b_f + 7.4)\sqrt{2 E_f t_f G_f} \tag{12}$$

$$G_f = 0.514 f_{co}^{\,0.236} \tag{13}$$

where P_{max} is debonding load, β_l is the bond length factor ($\beta_l = 1$ when $L \geq L_e$), b_f is the width of FRP, b_c is the width of concrete, t_f is the thickness of FRP, E_f is the elastic modulus of FRP, G_f is the interfacial fracture energy, β_w is the width ratio of FRP-concrete, and f_{co} is the compressive strength and f_t is the tensile strength of concrete.

For FRP-to-PC joints, an equation can be presented to predict the interfacial fracture energy based on the model proposed by Lu et al. [39]. A calibration factor of 0.165 was determined from the testing results as follows:

$$G_{f(PC)} = 0.165\beta_w^{\ 2}\sqrt{f_t} \tag{14}$$

The volume fraction of steel fibres has a significant effect on the interfacial fracture energy for SFRC. Therefore, the fibres-reinforcing index ($\frac{V_f L_f}{\phi_f}$) should be a factor determining the interfacial fracture energy. The comparison between the experimental and predicted interfacial fracture energy is presented in Figure 10. After regression analysis, the relationship between the interfacial fracture energy of FRP-SFRC and the fibres-reinforcing index is presented in Equation (15) as follows (R^2=0.93):

$$G_{f(SFRC)} = 4.533\left(\frac{V_f t_f}{\phi_f}\right)^{0.691} G_{f(PC)} \tag{15}$$

Based on the bond strength models of Lu et al. [39] and , the debonding loads of FRP-to-PC joints are determined by the interfacial fracture energy (G_f) and FRP stiffness ($E_f t_f$). A calibration factor of 1.68 obtained from the experimental results was introduced into the following equation:

$$P_{(PC)} = 1.68 b_f \sqrt{2E_f t_f G_{f(PC)}} \tag{16}$$

The bond strength between FRP and SFRC increased with the rising fibres volume due to the improved interfacial fracture energy. Two factors mainly affect the debonding loads, i.e. interfacial fracture energy and FRP stiffness, and the comparison of the predicted results is shown in Figure 11. By substituting the fibres-reinforcing index ($\frac{V_f L_f}{\phi_f}$) and the interfacial fracture energy (G_f) of the FRP-to-SFRC into Equation (9), the debonding force can be expressed as follows (R^2=0.93):

$$P_{(\mathrm{SFRC})} = (1.47 - 0.452\frac{V_f L_f}{\phi_f})b_f\sqrt{2E_f t_f G_{f(\mathrm{SFRC})}} \quad (0 \le \frac{V_f L_f}{\phi_f} \le 3.25) \quad (17)$$

4.3 Interfacial peak shear stress and slip

Theoretical models have been developed to predict the maximum interfacial shear stress (τ_{max}) and the corresponding slip (s_o). The peak shear stress and the corresponding slip are important factors determining the bond-slip relationship. For bilinear bond-slip models, it can be characterized by three factors, including the peak shear stress, the maximum slip at the peak shear stress, and the ultimate slip, which can be extracted from the non-linear bond-slip curves. Some bond-slip models have been developed and stated that the maximum shear stress is independent of FRP stiffness $E_f t_f$ [45, 56]. Wu and Jiang [45] proposed an accurate bond stress model $\tau_{max} = 1.31 k_w^2 \left(f_{co}\right)^{0.19}$ by considering the compressive strength of concrete (f_{co}) and width ratio (k_w). Similar to the model by Nakaba et al. [49], Dai et al. [35] consider the compressive strength in the model. Therefore, the models by Dai et al. [35] is not suitable for predicting the bonding behaviour of BFRP-SFRC. Similar to the model by Lu et al. [39], Sun et al. [34] consider both width ratio of FRP-to-concrete and the tensile strength in the model. However, the model by Sun et al. [34] is more complicated than the model by Lu et al. [39] and the calibration factors are difficult to be determined.

The model proposed by Lu et al. [39] is presented below:

$$\tau_{max} = 1.5\beta_w f_t \quad (18)$$

$$s_o = 0.0195\beta_w f_t \quad (19)$$

The model proposed by Sun et al. [34] is presented below:

$$\tau_{\max} = 1.35 + 0.25\beta_w f_t + 0.62 f_t \quad (20)$$

$$s_o = 0.016 - 0.0046\beta_w f_t + 0.11\beta_w \quad (21)$$

where τ_{max} is the peak shear stress, s_o is the slip at the peak shear stress, β_w is width ratio of FRP-concrete, and f_t is the tensile strength of concrete.

The comparison of peak shear stress is made between the predictions and the experimental results, as presented in Figure 12. For FRP-to-PC joints, the interfacial shear stress is determined by the width ratio (β_w) between FRP and concrete and the tensile strength of concrete (f_t). An equation was proposed based on the model developed by Lu et al. [39]. The peak interfacial shear stress can be written as:

$$\tau_{m(PC)} = 0.69\beta_w f_t \tag{22}$$

Due to the effect of the volume fraction (V_f) of steel fibres, the fibres-reinforcing index ($\frac{V_f L_f}{\phi_f}$) is introduced into the following equation as follows (R^2=0.92):

$$\tau_{m(\mathrm{SFRC})} = 1.836\left(\frac{V_f L_f}{\phi_f}\right)^{0.212} \tau_{m(PC)} \tag{23}$$

The comparison is made between the slip at the predicted peak shear stress and the experimental results, as presented in Figure 13. Based on the bond-slip model proposed by Lu et al. [39], the width ratio (β_w) of FRP-to-concrete and the tensile strength of concrete (f_t) are the two main factors influencing the slip at the peak shear stress. For FRP-to-PC joints, the slip at the peak shear stress can be expressed as follows:

$$s_{o(PC)} = 0.011\beta_w f_t \tag{24}$$

The fibres volume has a significant effect on the slip at the peak shear stress for SFRC as the slip increased with the fibres volume. Thus, the fibres-reinforcing index ($\frac{V_f L_f}{\phi_f}$) is considered when predicting the slip. An equation is proposed based on the regression analysis as follows (R^2=0.90):

$$s_{o(SFRC)} = 2.651(\frac{V_f t_f}{\phi_f})^{0.439} s_{o(PC)} \quad (25)$$

4.4 Interfacial bond-slip relationship

The interfacial bond-slip curve presents the relationship between the local interfacial shear stress and the local slip, which can be used to analyse the bond performance of FRP-strengthened concrete structures using analytical and numerical methods. In this study, the nonlinear bond-slip curves can be experimentally obtained by using the DIC method, as shown in Figure 14. Two distinctive branches can be identified, i.e. ascending branch and descending branch. The shear stress increases up to the peak shear stress (τ_{max}) with the shear slip. After reaching the peak stress, the interfacial shear stress decreases with the increase of the shear slip. Popovics' equation [57] can be used to predict the interfacial bond-slip as follows:

$$\tau(s) = \tau_{max}[\frac{s}{s_o}\frac{n}{(n-1)+(s/s_o)^n}] \quad (26)$$

where $\tau(s)$ is the local shear stress, s is the local slip, τ_{max} is the peak local shear stress, s_o is the slip at the peak shear stress, and n is the parameter depending on the concrete compressive strength which determines the slope of both the ascending and descending branches [57]. The analytical results of the peak shear stress (τ_{max}) and the corresponding slip (s_o) can be obtained by Equations (23) and (25). The interfacial bond-slip relationship between PC and SFRC is similar as the specimens with different fibres volumes exhibit similar shapes. The slopes of the ascending branch and the descending branch of all the specimens are almost the same. Therefore, the coefficient (n) should be a constant in this study. Based on the experimental results, the constant (n) ranges from 2.827 to 2.926. The average value of n is 2.888 in this study. In addition, the interfacial fracture energy (G_f) increases with the volume fraction of steel fibres as the peak shear stress and the corresponding shear slip increase with the volume fraction of steel fibres, which is shown in Figure 14 (e).

5. Conclusions

This study investigates the effect of short steel fibres on the interfacial bond behavior, i.e. the bond strength, the peak shear stress, the corresponding slip and the bond-slip relationship between BFRP and SFRC. The single-lap shear testing method was used to conduct the experimental study. The digital image correlation (2D-DIC) technique was employed to measure the full fields of displacement and strain. The following conclusions can be drawn based on the experimental results:

1. Adding short steel fibres has no effect on the failure mode as debonding occurred in the concrete layer for all the specimens. Pull-out of steel fibres from the concrete matrix was observed when debonding occurred;
2. Bond strength increases with the volume fraction of short steel fibres. A calibrated bond strength model is proposed for predicting the bond strength between BFRP and SFRC, which fits very well with the experimental results;
3. The effective bond length can be obtained by using either the longitudinal strain fields or the strain distribution gradient of the BFRP sheets, which yields similar results. The effective bond length decreases with the increasing fibre volume;
4. The interfacial shear stress and the corresponding slip at peak shear stress increase with the fibre volume. The calibrated models of the peak shear stress and the corresponding slip are proposed by incorporating the effect of short steel fibres, which matches well with the experimental results;
5. The interfacial fracture energy is significantly affected by the short steel fibres as the area of the interfacial bond-slip curves of BFRP-to-concrete increases with the increase of fibre volume;
6. The bond-slip model between BFRP and SFRC is proposed based on Popovics' equation, which matches well with the experimental results and the derived coefficient

correlation values.

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