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Effect of Aggregate Size on Bond Behaviour between Basalt Fibre Reinforced Polymer Sheets and Concrete

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Abstract

The effect of aggregate size on the interfacial bond behaviour between BFRP sheets and concrete is investigated in this study by conducting single-lap shear tests. The effect of aggregate sizes (i.e. 5-10 mm, 10-15 mm, and 15-20 mm) on the debonding load, maximum bond stress, effective bond length, local slip at peak shear stress, as well as the bond-slip relationship between the BFRP sheets and concrete are presented and discussed. The experimental results have shown a significant effect of the aggregate size on the interfacial bond-slip behaviour. The interfacial shear stress decreased when the aggregate size increased due to the decreased tensile strength of concrete. The relative slip between BFRP and concrete at the peak bond stress increased with the increasing aggregate size also because of the reduced tensile strength of concrete. Existing models regarding the bond strength and interfacial bond-slip are adopted and recalibrated against the experimental results in which the size effect of aggregates is incorporated.

Keywords: Basalt fibre reinforced polymer (BFRP): Bond-slip; Debonding; Mechanical testing.

1. Introduction

Fibre reinforced polymer (FRP) has been increasingly used in strengthening reinforced concrete (RC) structures [1, 2]. FRP including Glass FRP, Carbon FRP and Basalt FRP has excellent characteristics, such as light weight, high fatigue performance, high strength to weight ratio, superior resistance to corrosion, and cost effectiveness [3]. The most common premature failure of externally bonded (EB) FRP sheets strengthened beams/slabs is the debonding of the FRP sheets from a concrete substrate due to high stress concentrations [4]. Premature debonding limits the strengthening efficiency of FRP strengthened structures since it occurs at lower FRP strain than its ultimate strain [5]. The interfacial bond behavior between FRP and concrete is a critical factor in controlling failures of FRP-strengthened concrete structures [6, 7]. The factors determining the interfacial bond capacity include mechanical properties of concrete substrates, mechanical properties of adhesive, and stiffness of FRP [8-10].

FRP has been widely used in the shear or flexural strengthening of RC beams under different loading conditions [11-13]. It is found that debonding is a dominant failure mode that is induced by a localized flexural or shear-flexural cracks initiated in the concrete. A crack may develop within the adhesive layer and then through the layer of FRP reinforcement due to shear and peeling stress [14]. Pan et al. [15, 16] conducted experimental study on the effect of aggregate content ranging from 0.030 to 0.119 on the FRP/concrete bond capacity, where the aggregate content is defined as the area fraction ratio of coarse aggregates to the total area of the roughed concrete surface. The ratio of cement to water to sand to aggregate was different in each mixing proportion and the aggregate sizes for each mixing were the same, ranging from 4.75 mm to 20 mm. The results showed that the ultimate bond strength increased with the aggregate content. The initial debonding strength, the residual shear strength, and the maximum slippage between FRP and concrete were affected by the aggregate content. In addition, the interfacial fracture energy was affected by the interfacial shear interlocking and softening as aggregate interlocking and abrasion effects were sensitive to the aggregate content. A thin layer of concrete with 2-5 mm thickness is usually attached to FRP sheets when debonding occurs due to the fracture of concrete layer [17].

Numerous studies have investigated the aggregate size effect on the mechanical properties of plain concrete. It was found that the aggregate type and size are important parameters in the formation of interfacial transition zone (ITZ) and subsequently in micro-crack formation and propagation [18-20]. Tülin et al. [20] conducted an experimental study and found that tensile strength decreased as the aggregate size increased. This is because larger aggregates resulted in an increased interfacial transition zone (ITZ) and increases micro-cracks in the vicinity of the aggregate. In addition, larger aggregates resulted in a zone of poor bond in concrete due to the internal bleeding where higher tensile stress is concentrated, indicating that increasing aggregate size leads to a lower bond strength [21]. On the other hand, Özturan et al. [22] stated that the compressive strength of concrete was mainly dominated by the quality of mortar and surface characteristics of aggregates rather than aggregate type and size. The fracture energy and fracture toughness related to the mechanical properties of concrete have been widely studied [23-27]. As concluded from these studies, both the fracture energy and fracture toughness increased with the aggregate size. This is because cracking likely propagates along the weaker interfacial zone in concrete upon loading. The interfacial toughness of aggregates is lower than the matrix, the advancing crack is prone to deflect the aggregate, resulting in a tortuous cracking path and more energy needed [27]. Hu and Wang [28] studied the effect of coarse aggregates on concrete rheology through infinite slope stability analysis, and found that larger aggregates resulted in a lower yield stress and viscosity of concrete as the larger aggregates had a larger internal friction angle.

In the literature, Pan et al. [15] investigated the effect of the aggregate content on the bond behaviour between FRP and concrete. However, no study has been carried out to investigate the effect of aggregate size on the interfacial bond behaviour of FRP-concrete. Therefore, in this study the experiments were conducted to investigate the effect of coarse aggregate size on the bond behaviour between FRP and concrete by using the single-lap shear testing method. The digital image correlation (2D-DIC) technique was used to measure the full-field displacements and strains of the specimens. The bond-slip curves of the specimens can be experimentally obtained from the FRP strain distributions during the loading process. In addition, the effects of aggregate size on the interfacial bond strength, maximum bond stress, effective bond length, and local slip at peak bond stress and bond-slip relationship are also examined.

2. Experimental program

2.1 Material properties

The effect of coarse aggregate size on the bonding behavior was investigated in the test program and the coarse aggregate sizes (d_{n}) with three ranges from 5 - 10 mm, 10 - 15 mm to 15 - 20 mm were used as shown in Figure 1. In addition, the concrete mix design was based on 5 - 10 mm coarse aggregate size and two different concrete mixes with the grade of 40 MPa and 60 MPa were used in this experiment. The details of concrete mix design are summarized in Table 1.

In the tests, concrete prisms with 350 (L) x 150 (H) x 150 (W) mm as substrate were demolded 24 hours after casting and then cured in water tanks at room temperature for 28 days. The mechanical properties of concrete with different coarse aggregate sizes, including compressive strength $f_c{}'$ and splitting tensile strength f_t were measured to study the effect of aggregate size on the bond behaviour. Three concrete cylinders with the diameter of 100 mm and the height of 200 mm from each batch were tested to obtain the compressive strength according to ASTM C39 [29]. Three concrete cylinders with the diameter of 150 mm and the height of 300 mm were tested for the splitting tensile test according to ASTM C496 [30]. The testing setups are shown in Figure 2 and the mechanical properties of four groups of concrete specimens are summarized in Table 2.

The adhesive used to saturate the fibre was a mixture of epoxy resin and hardener at a ratio of 5:1. The epoxy resin has an ultimate tensile strength of 50.5 MPa, elastic modulus of 2.8 GPa and rupture tensile strain of 4.5%. Unidirectional basalt fiber sheets with a unit weight of 300 g/m^2 and nominal thickness of 0.12 mm were used in this study. BFRP coupon tensile tests based on ASTM (2008) [31] were conducted to determine the material properties of the BFRP sheets. The tensile strength, Young's modulus, and rupture strain of the BFRP sheet are 1,333 MPa, 73 GPa and 1.88%, respectively.

2.2 Specimens details

A total of 12 specimens were tested in this study. The surface of all the specimens were roughened by a needle scaler to expose coarse aggregates. Manual lay-up procedure was conducted to bond the BFRP sheets onto the surface of concrete substrates. Two layers of BFRP sheets with the width of 40 mm were bonded with adhesive on one side of the concrete prism along the axial direction. All specimens had a bonded length of 200 mm, which was longer than the effective bond length estimated from the previous model [32]. An unbonded length of 50 mm was reserved to eliminate the effect of concrete edge during the loading process [33], as shown in Figure 3. The specimens were cured for 7 days in order to ensure full hardening of epoxy.

Table 3 gives the details and testing results of the 12 specimens. The specimen ID was assigned to each specimen as "GX_CY_d_n". "GX" means the testing group from G1 to G4, totally four testing groups in this study. "*CY*" refers to the grade of concrete, and C40 and C60 represent the concrete prisms with the compressive strength of 40 MPa and 60 MPa, respectively. The letter "*d*" refers to the size of coarse aggregate (e.g. d_n (5-10) means the aggregate size ranging 5-10 mm). The letter "*n*" represents the specimen number from 1 to 3 (three identical specimens for each configuration).

2.3 Testing setup

Figure 3 shows the setup of the single-lap shear testing. All the specimens were tested under displacement control at a loading rate of 0.3 mm/min [34]. The testing machine has an inbuilt load cell to measure the load during the test. Two strain gauges with 5 mm gauge length were mounted on the surface of BFRP sheets to measure the axial strain during the test. One linear variable differential transducer (LVDT) with a range of ±10 mm was used to measure the displacement of BFRP sheets. A camera together with digital image correlation (DIC) technique was used to monitor the strain distribution of the BFRP sheets for all the tests. Three specimens were tested for each configuration to reduce the uncertainties of experimental results.

3. Test results and discussions

3.1 Failure mode

Failure mode determines the performance and efficiency of the bonding between BFRP sheets and concrete. There was only one failure mode in this study, i.e. debonding failure in the concrete substrate, where a thin layer of concrete was attached to the BFRP sheets after debonding. In addition, the debonding failure initiated at the loaded end for all the specimens, which was consistent with the previous studies [17, 35]. The typical debonding failure mode of the specimens after testing is shown in Figure 4. It was observed that the aggregate size had a limited effect on the failure for all the specimens. To examine the distribution of aggregates, the method of image thresholding was employed and the black area and the white region represent the aggregates and the mortar, respectively. It was observed that the small aggregate size (i.e. 5-10 mm) resulted in more uniform and denser distribution than that of large aggregate sizes. As shown in Figure 5 (a), small aggregates shown in black were attached with FRP after debonding. The pull-out of the small aggregates from the concrete matrix can be seen for specimen G1_C40_5-10_1. In contrast, more mortar is attached with BFRP sheets for the specimen G3_C40_15-20_1 after debonding, as shown in Figure 5 (c). It was also observed that the specimens G3_C40_15-20 with the largest aggregates in the adhesive-concrete layer experienced fracture of mortar with pull-out of small amounts of aggregates.

3.2 Load and displacement

For the specimens with the same concrete mix but different coarse aggregate sizes, the load and displacement curves are plotted in Figure 6. It can be seen that the debonding loads reduced with the increase of the aggregate size. In addition, two different concrete grades of C40 and C60 with the same aggregate size 5 - 10 mm were prepared in this study. The testing results of C60 with the aggregate size of 5-10 mm were shown in Figure 7. The debonding load increased with the tensile strength, which is consistent with the previous studies [36, 37]. The measured displacement includes the shear slip of the bonded part and the elongation of the unbonded part of the BFRP sheets [17]. The average debonding loads for specimens G1_C40_5-10, G2_C40_10-15, and G3_C40_15-20 were 11.7 kN, 10.9 kN, and 10.3 kN, respectively. The debonding loads decreased slightly with the rising maximum aggregate size, which indicates the aggregate size has effects on the interfacial bond strength. In addition, four points (i.e. A-D, E-H and I-L as shown in Figure 6) were selected from the load-displacement curves of the specimens G1_C40_5-10_2, G2_C40_10-15_3, and G3_C40_15-20_3, respectively in order to track the strain distributions and the interfacial shear stress distributions at different loading stages.

In this study, the mass ratio of coarse aggregates over total weight was kept the same at approximately 40%. Figure 8 shows the relationship between the aggregate size and the aggregate interlocking action. For the specimens with smaller size aggregates, the spacing between each aggregate is small due to the fact that the small aggregates are densely distributed, which results in strong interfacial shear interlocking action. Meanwhile, for the specimens with larger aggregate size, more spaces between each aggregate result in relatively weak interlocking action and large sized aggregates cannot be easily pulled out from the matrix since the deep embedment depth gives sufficient bond and friction. It should be noted that the surface treatment method was surface chiseling in order to remove the weak layer of paste and expose the aggregates for a stronger bonding. The interfacial shear interlocking is a major factor affecting the debonding failure of FRP as the aggregate interlocking action is very sensitive to the aggregate as reported in the previous study [15]. Stronger interlocking action results in a higher interfacial bond strength between FRP and concrete as higher fracture energy is required to develop cracks and pull-out of the coarse aggregates. This is because the tensile strength of concrete is a key factor determining the interfacial bond strength of FRP-concrete and increasing aggregate size leads to a lower tensile strength. This is because the increased surface area of large size aggregate results in an increasing stress concentration and micro-cracks in the vicinity of the aggregates [20]. The lower tensile strength of concrete results in a weaker interfacial bond strength of FRP-concrete as the bond strength is proportional to the tensile strength of concrete [36].

Figure 9 plots the typical load-displacement curve for shear bond tests. Theoretically, three stages exist before the complete debonding, i.e. elastic stage, softening stage, and debonding plateau. After reaching elastic stage, microcracks initiate at the adhesive-concrete interface with the increase of shear slip [31]. Debonding initiates at the loaded end when approaching the end of the softening stage. Then a plateau can be seen with the growth of the displacement, illustrating the gradual debonding process. The debonding plateau stage is mainly dominated by the bond length of the BFRP sheets, a longer debonding plateau can be found when using a longer bond length of FRP as reported in the previous study [35]. In this study, a bond length of 200 mm was used and it is long enough to develop the debonding plateau [38].

3.3 Strain distribution

The strain distributions of all the specimens are shown in Figure 10. The strain derived from DIC has been compared with the results from strain gauges. It can be observed that there was a significant spatial variation in the axial strain along the surface of BFRP sheets. The fluctuations in the measured surface strain were induced by the local material variations and the material in-homogeneities due to the non-uniform distributions of resin and the varied thickness of FRP sheets [39-41]. To eliminate the influence of the local material variations, a nonlinear regression analysis can be performed by using Equation (1) [39] to fit the strain:

$$\varepsilon(x) = a + \frac{b}{1+(\frac{x}{x_o})^{c}} \tag{1}$$

where a, b, c and x_o are the coefficients to be obtained from testing results and x is the distance from the loaded end. The original DIC strain distributions and the fitted strain distributions are shown in Figure 10. It is noted that FRP strain derived from the DIC technique was verified against those directly measured by strain gauges with very high accuracy and this technique was also successfully used in the previous studies [40]. Each curve refers to the strain distribution along the FRP sheets at a particular loading stage, which is shown in Figure 6. The strain distribution presents a descending curve from the loaded end toward the free end of the BFRP sheet as indicated in Figure 10. The strain firstly increased with the rising applied load. After the initiation of debonding at the loaded end, a strain plateau can be found in the graphs, illustrating the stress transfer zone [40, 41].

Figure 10 shows that the peak strain decreases with the increase of the aggregate size. The ultimate strain for specimens G1_C40_5-10_2, G2_C40_10-15_2, and G3_C40_15-20_3 were 1.40%, 1.29%, and 1.22%, respectively. This meant that the aggregate size had a significant effect on the BFRP strain within the bonded region. After the softening stage, more micro-cracks were accumulated to form a destruction crack within the layer of adhesive-concrete. Due to the action of aggregate interlocking, the BFRP sheets continued to resist the shear force. The specimens with smaller aggregate size possessed a higher fracture energy due to the stronger interlocking action. The specimens G1_C40_5-10 had the largest strain among the three groups. It is because the larger shear force resulted in larger deformation of the BFRP sheets with the same stiffness.

3.4 Bond stress and local slip calculation

The interfacial shear stress distribution along the bonded length reflects the stress development and stress transfer in the interface between BFRP sheets and concrete. The bond-slip laws in longitudinal direction can be obtained from the FRP strain by using Equation (2). The interfacial shear stress distribution within the bond length can be evaluated by imposing the equilibrium condition of a FRP sheet with a length *dx* bonded to concrete [42, 43], as:

$$\tau(x) = t_f E_f \frac{d\varepsilon_f}{dx} \tag{2}$$

where τ is the interfacial shear stress, $\frac{d\varepsilon_f}{dx}$ 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 sheet and concrete at distance x from the free end of the specimen can be calculated by assuming a zero slip at the free end as [43]:

$$s(x) = \int_0^\infty \varepsilon_f dx \tag{3}$$

The fitted strain distribution measured from the DIC technique can provide consecutive values, which can reduce the data intervals. This is beneficial for the accuracy of the interfacial shear stress and local slip. Figure 11 shows the interfacial shear stress distributions along the length of the BFRP sheets at different loading stages. The interfacial shear stress distributions for all the specimens were similar as the specimens with different sizes of aggregates exhibited the same shapes. As the applied load increased, the maximum interfacial shear stress moved along the BFRP sheets from the loaded end, which implied debonding crack propagation. Theoretically, the interfacial shear stress should be constant during the loading process while the experimental results presented stress fluctuations, as also observed by previous studies [35, 44]. The possible reason is that the length of the interfacial shear stress transfer zone increased during the loading process, which can be evidenced by the interfacial shear stress distributions in Figure 11. It should be noted that the transfer zone of interfacial shear stress can be defined from the interfacial shear stress distributions [38].

The peak shear stress (τ_m) for all the specimens are summarized in Table 4. The results indicate that the aggregate size has significant influences on the interfacial shear stress. The shear stress decreased with the increasing aggregate size. For the specimens G1_C40_5-10, G2_C40_10-15, and G3_C40_15-20, the average values of the shear stress were 6.23 MPa, 5.08 MPa, and 4.77 MPa, respectively. These shear stresses of specimens G2_C40_10-15 and G3_C40_15-20 result in a reduction of 18% and 23% when respectively compared to specimens G1_C40_5-10. In addition, as shown in Table 3 the slip s_o increased from 0.112 to 0.125 and 0.136 when the aggregate size increased from 5-10 to 10-15, and 15-20 mm, respectively. The tensile strength of concrete should be a key factor governing the interfacial bond of FRP-concrete interface as debonding occurred inside the concrete layer in this study. The tensile strength of the concrete substrates decreases with increasing the aggregate size [20]. As can be seen that increasing the aggregate size leads to a reduction in the interfacial shear stress. This observation is reasonable since using larger aggregates leads to a reduction in the tensile concrete strength and thus the interfacial shear stress.

3.5 Effective bond length

Effective bond length is the bond length beyond which no further increase in the ultimate load can be achieved [4]. This can be evidenced by the load and displacement curves as well as the debonding plateau after the initial debonding load. An active bond zone exists at any stage of loading and over which interfacial shear stresses are transferred from the fibre sheet to the concrete, which is consistent with the finding in the previous studies [45, 46]. In this study, the effective bond length can be extracted from the strain distributions as it is defined through the strain distributions where the effective bond length is the length required for the strain to vanish [47, 48].

The length of the active zone at debonding loads can be evaluated using longitudinal strain fields of the BFRP sheets obtained from the DIC analysis as shown in Figure 12. Successive digital images were captured and analysed using the DIC technique, and longitudinal strain field corresponding to each load level was derived. As can be seen from the figure that the effective bond length increased with the aggregate size. The effective bond lengths for specimens G1_C40_5-10_2, G2_C40_10-15_3, and G3_C40_15-20_3 were 34 mm, 41 mm, and 52 mm, respectively. The average effective bond length for groups G1_C40_5-10, G2_C40_10-15, and G3_C40_15-20 were 37 mm, 45 mm, and 54 mm, respectively. The effective bond length increased with the aggregate size while it is inversely proportional to the tensile strength of concrete [37]. This statement is reasonable because using larger aggregates leads to a reduction of the tensile strength of concrete and thus results in longer effective bond length. In addition, as observed from the strain contours of Figure 12, the strain distribution of the specimen G3_C40_15-20_3 was not uniform as compared with G1_C40_5-10_2. This is because the large aggregates in the adhesive-concrete layer are not placed uniformly and closely with each other as compared with the small aggregates. In addition, the interfacial shear stress for the interface of FRP-aggregate and FRP-mortar is different, which results in non-uniform strain distributions in the bonded area. This variation became more prominent with specimens G3_C40_15-20 with 15-20 mm large aggregates.

4. Theoretical predictions and proposed models

4.1 Mechanical properties of concrete with various aggregate sizes

To investigate the bond behaviour between FRP and concrete, the tensile strength of concrete considering the aggregate size effect needs to be determined. In addition, the tensile strength of concrete can be estimated from its compressive strength. As a result, this section proposes new empirical equations to predict the compressive and tensile strengths of concrete in which the effect of the aggregate size is taken into consideration. As shown in Figure 13, the compressive strength increases while the tensile strength decreases with increasing the aggregate size. The results are consistent with the previous study [20]. This is because larger aggregates result in an increased interfacial transition zone (ITZ) and increases of micro-cracks in the vicinity of the aggregate. In addition, larger aggregates result in poor bond zone inside concrete due to the internal bleeding [20, 28]. Based on Bazant's law of size effect [49] and the calibrated model by Kim et al. [50], Jiang and Wu [51] proposed a model to predict the unconfined concrete uniaxial strength by considering the aggregate size effect:

$$f_c = f_c^{'} \cdot \delta(d_{max}, h, d_a{}^{\mathrm{m}}) \tag{4}$$

$$\delta(d_{max}, h, d_a{}^{\mathrm{m}}) = \alpha + \frac{B}{\sqrt{1 + \frac{d_{max}}{\lambda_o d_a{}^m}(h/d - \beta)}} \tag{5}$$

where $f_c^{'}$ is the strength of concrete specimen of standard size, f_c is the actual strength of concrete specimen considering the size effect, *h* and *d* are the height and diameter of specimens, respectively, d_{max} is the maximum aggregate size of concrete, $d_a^m \approx 1$based on the regression results of Kim et al. [50], α, B, λ_o, m, and β are the coefficients which can be determined by the regression of testing results. It should be noted that the height and diameter of the concrete cylinder in this study are h = 200 mm and d = 100 mm, respectively. The compressive strength of concrete considering the aggregate size can be expressed as follows:

$$f_c = \alpha f_c^{'} + \frac{B f_c^{'}}{\sqrt{1 + \frac{d_{max}}{\lambda_o}(\frac{h}{d} - \beta)}} \tag{6}$$

Based on the standards ACI 318-14 [52] and CEB-FIB [53], the splitting tensile strength of concrete can be correlated with compressive strength by the following equation:

$$f_t = \varphi (f_c)^c \tag{7}$$

where f_t (MPa) is the predicted splitting tensile strength of concrete, f_c (MPa) is the predicted compressive strength of concrete, $f_c^{'}$ is the designed compressive strength that was 40 MPa in this study, and d_{max} (mm) is the maximum aggregate size. Given a set of testing data in Table 2 (i.e. f_c and d_{max}), coefficients α = 1.568, B = -1.136, λ_o = 1.933, and β = 1.415 can be obtained by using the Matlab (2016a) curve fitting toolbox.

Equations (8) and (9) can be used to describe the relationship between the concrete strength and the maximum aggregate size. As shown in Figure 13, the predicted compressive strength and splitting tensile strength show good agreement with the experimental results.

$$f_c = 1.568 f_c^{'} - \frac{1.136 f_c^{'}}{\sqrt{1 + \frac{d_{max}}{1.933}(\frac{h}{d} - 1.415)}} \tag{8}$$

$$f_t = 7845 (f_c)^{-2.006} \tag{9}$$

4.2 Interfacial bond strength

In order to predict the ultimate debonding load between FRP and concrete, numerous studies have been conducted to develop bond strength models based on empirical data and theory of fracture mechanics. The bond strength can be calculated from the FRP stiffness and interfacial fracture energy. As the same BFRP sheet has been used in this study, the bond strength is mainly dominated by the interfacial fracture energy. In this study, the bond length of BFRP sheets was 200 mm, which was long enough to develop the effective bond length [38]. This is evidenced by the debonding plateau in the load versus displacement curves. Based on the fracture mechanics, two models from CNR DT-200 [54] and Lu et al. [37] are employed to predict the interfacial fracture energy and the predicted results are presented in Table 4. CNR DT-200 [54] presented a formula to calculate the fracture energy of the FRP-concrete interface, which can be described as:

$$G_f = k_G k_b \sqrt{f_c^{'} f_t} \tag{10}$$

where G_f(N/mm) is the interfacial fracture energy, $f_c^{'}$ is the cylinder axial compressive strength of concrete, f_t is the tensile strength of concrete, k_G is the fracture energy coefficient with an average value of 0.064, and k_b is a geometrical factor, which can be expressed as,

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

Lu et al. [37] also provided the following formula for calculating the interfacial fracture energy, as:

$$G_f = 0.308 \beta_w^{2} \sqrt{f_t} \tag{12}$$

$$\beta_w = \sqrt{\frac{2.25 - b_f / b_c}{1.25 + b_f / b_c}} \tag{13}$$

where f_t is the tensile strength of concrete, b_f and b_c are the width of FRP and concrete, respectively, and β_w is the width ratio between FRP and concrete.

As can be seen from Table 4, the two models above cannot predict well the interfacial fracture energy of concrete with varying aggregate size. It is because the aggregate size is not considered in the empirical models. This study is aimed to achieve more accurate predictions on debonding by considering the effect of aggregate size. In this study, the debonding loads decreased with rising the maximum aggregate size. The CNR DT-200 [54] model considers both compressive strength and tensile strength of concrete. The calculated interfacial fracture energy underestimates the experimental results. Therefore, CNR DT-200 [54] model cannot accurately predict the interfacial fracture energy. Based on the existing interfacial fracture energy models in the literature, it can be found that the interfacial fracture energy (G_f) correlates well with the tensile strength (f_t) of concrete and the width ratio (β_w) between FRP and concrete. In this study, Lu et al. [37] model was recalibrated to predict the experimental results. The interfacial fracture energy can be described by the function of f_t^d and β_w^2 [37], as given in Equation (14). Two coefficients γ and d can be obtained through fitting procedure based on the testing data.

$$G_f = \gamma \beta_w^{\ 2} f_t^d \qquad (14)$$

After fitting analysis of the testing results, two coefficients γ and d are determined as 0.420 and 0.695, respectively, as shown in Figure 14. β_w is the width ratio between FRP and concrete which can be calculated by Equation (13). In addition, Equation (15) can be used to predict the interfacial fracture energy in consideration of the maximum aggregate size. Also, the mean value of the predictions based on the proposed model provides acceptable accuracy, as given in Table 4.

$$G_f = 0.420\beta_w^{\,2} f_t^{0.695} \tag{15}$$

The calibrated bond strength model is employed to calculate the debonding loads. Fracture energy obtained by Equation (15) was substituted into Equation (16) to predict the debonding loads, as given in Table 5. A calibration factor $\eta = 1.212$ was introduced herein to consider the effect of the maximum aggregate size.

$$P = \eta b_f \sqrt{2E_f t_f G_f} \tag{16}$$

Figure 15 shows the experimental and predicted debonding loads. The points (i.e. red, blue, and pink) located above the baseline ($y=x$) indicate the under-predictions of the debonding load. The proposed model by incorporating the effect of aggregate size fits very well with the experimental results as the correlation coefficient R^2 is 0.891 and the mean value of P_{pre}/P_{exp} is 1.001 (S.D. = 0.028).

4.3 Peak interfacial shear stress

Many analytical models have been developed to predict the interfacial shear stress between FRP and concrete [10, 37, 55-58]. Six interfacial shear stress models were selected in this study to compare their predictions with the experimental data, as shown in Figure 16. Different parameters (e.g. concrete tensile strength f_t, width ratio of FRP-to-concrete β_w, concrete compressive strength f_c', elastic modulus of FRP E_f, thickness of FRP t_f, and bond length of FRP L) were considered in each model. The integral absolute error (IAE), which has been often used for model assessments, is employed herein to evaluate the accuracy of the existing models of peak interfacial shear stress, as presented in Equation (17) [38, 59].

$$IAE = \sum \frac{|Expe. - Theo.|}{|Expe.|} \tag{17}$$

where *Expe*. and *Theo*. are the experimental and theoretical results, respectively.

The higher IAE value indicates that the theoretical model cannot well predict the interfacial shear stress. The predicted results obtained by Ko et al. [55] and Sato et al. [56] are based on the compressive strength of concrete with higher IAEs. The predicted results obtained by Tanaka [57], Neubauer and Rostasy [10], Yang et al. [58], and Lu et al. [37] are based on the tensile strength of concrete. Among these models, the model by Lu et al. [37] can generate the most accurate predications due to the lowest mean value of IAE. Based on the existing bond stress models, the interfacial shear stress can be described by the function of β_w and f_t^e [37], as given in Equation (18). The coefficients k and e determined from the fitting analysis are 0.694 and 1.396, respectively, as shown in Figure 17.

$$\tau_m = \kappa \beta_w f_t^e \tag{18}$$

The predicted peak shear stress obtained from Equation (19) matches well with the experimental results as its mean value is 0.982 (S.D. = 0.042), as given in Table 6. The predicted interfacial shear stress decreases with the increase of the maximum aggregate size, which is evidenced by the experimental results.

$$\tau_m = 0.606 \beta_w f_t^{1.396} \tag{19}$$

4.4 Slip at peak shear stress

The slip s_o is the relative displacement between FRP sheet and concrete at the peak interfacial shear stress, which is an important parameter for analysing shear softening in the debonded zone. Numerous bond-slip models have been developed in the literature [6, 37, 59, 60]. There are two branches existing in these models, namely the ascending branch and the descending branch, respectively. During the elastic stage and softening stage, the stress keeps increasing to the peak stress (τ_m). Debonding stage initiates in the concrete layer with increasing the shear slip. In the existing bond-slip models, the slip s_o can be predicted by the equations in Table 7.

The accuracy of each analytical model is evaluated by comparing the experimental results with the predicted results. The predicted slip by using the previous models by Nakaba et al. [7] and Neubauer and Rostasy [10] is a constant value, which is different from the testing results. The model proposed by Lu et al. [37] shows a higher IAE as compared with the model by Sun et al. [60]. The model developed by Sun et al. [60] is the most accurate due to its lowest IAE. Based on the analytical models and the experimental results, the slip s_o is affected by the width ratio factor (β_w) and the tensile strength of concrete (f_t). Calibration is conducted to predict the slip at the peak bond stress based on the model developed by Sun et al. [60].

$$s_o = \omega - \zeta \beta_w f_t + \theta \beta_w \tag{20}$$

As can be seen from Figure 18, coefficients ω, ξ and θ can be obtained by the regression analysis. Based on Equation (21), the analytical slip at the peak shear stress presents good matches with the experimental results by giving the mean value of 1.029 (S.D. = 0.055), as given in Table 8.

$$s_o = 0.111 - 0.016 \beta_w f_t + 0.080 \beta_w \tag{21}$$

4.5 Interfacial bond-slip relationship

An interfacial bond-slip relationship is of fundamental importance in modelling FRP-strengthened RC structures. In this study, the interfacial shear stress and slip are obtained by analysing the surface strain in the BFRP sheets from the DIC technique at the centreline of the stress-transfer length [61, 62]. The bond stress can be obtained from the measured strain using Equation (2). The relative slip between BFRP and concrete can be obtained by integrating the strain profile. The previous studies [7, 43] stated that the assumptions should be made to define the slip distribution along the FRP sheets: (1) zero slip between concrete and BFRP at the free end of the BFRP sheet; (2) deformation of concrete specimen far from the external cover is negligible with respect to its BFRP counterpart; and (3) linear variation of strain in BFRP sheet. Non-linear bond-slip curves with an ascending branch and a descending branch based on the measured data can be obtained, as shown in Figure 19.

Popovics's equation [63] is used to predict the relationship between the interfacial shear stress and slip, as:

$$\tau = \tau_{\max}[\frac{s}{s_o}\frac{n}{(n-1)+(s/s_o)^{\mathrm{n}}}] \tag{22}$$

where τ is the interfacial shear stress, s is the local slip, τ_{max} is the peak interfacial shear stress, s_o is the slip at the peak shear stress, and n is a coefficient related to the concrete compressive strength, which causes the slope of both ascending and descending branches [63]. Coefficient n was proposed as a constant in some studies [7, 64]. However, the correlation between the coefficient n and the aggregate size can be found in this study as the compressive strength of the concrete substrates increases with the aggregate size. Table 9 gives the regression coefficient n and the corresponding correlation coefficient. Equation (23) developed by Popovics [63] is used to establish the relationship between n and the maximum aggregate size through the compressive strength of concrete. Equation (24) is proposed based on the experimental results to predict the coefficient n and the coefficient of correlation R^2 is 0.822, as shown in Figure 20. The prediction by Equation (24) shows a low mean value of 0.997 (S.D. = 0.011).

$$n=\alpha+\beta f_c \tag{23}$$

$$n=4.52-0.038f_c \tag{24}$$

Figure 21 shows the shear stress versus shear slip response for the interface between BFRP and concrete, in which the predictions match the experimental results well. There are three stages for the bond-slip curves. After linear elastic response at around 40% of the maximum shear stress, it is non-linear up to the peak stress with the increase of shear slip. In the descending branch after reaching τ_{max}, a softening stage induced by microcracks can be observed where increasing shear slip results in a decreasing shear stress. The shear stress gradually drops to zero with the increase of shear slip.

Similar shapes of the interfacial shear stress versus slip curves and the bond-slip curves were observed. The peak interfacial shear stress decreases with the increasing maximum aggregate size. In addition, the slope of the ascending branch decreases as the maximum aggregate size increases due to the decreased interfacial fracture energy. It should be noted that the area of the bond-slip is defined as the interfacial fracture energy. Popovics's equation can be used to predict the shear stress versus slip relationship of BFRP-concrete interface by considering coarse aggregate of different sizes as the prediction fit well with the experimental results.

As shown in Figure 21, the proposed model yields better predication than the two existing models with a higher accuracy and the correlation coefficient R^2 predicted by the proposed model are larger than 0.9 for all the specimens, as given in Table 9. Two existing bond-slip models by Nakaba et al. [7] and Dai and Ueda [65] cannot provide very accurate predictions as compared with the experimental results, as shown in Figure 21. It is because that different material might have been used and the effect of aggregate size was not incorporated into two existing models.

5. Conclusion

This study investigates the effect of aggregate size on the bond behaviour between BFRP and concrete, including the debonding load, maximum interfacial shear stress, and bond-slip relationship. The single-lap shear test method was utilized to conduct the experimental study. The 2D-DIC technique was employed to measure the full fields of displacement and strain. The following conclusions can be drawn:

1. Debonding of all the tested specimens occurred because of the failure of the concrete substrate. The pull-out of small aggregates from the concrete matrix was observed on the debonded BFRP sheets.
2. The debonding loads decreased with the increasing coarse aggregate size. Compared to the specimens with the aggregate size of 5-10 mm, a reduction of 6.55% and 10.04% for the specimens with the aggregate size of 10-15 mm and 15-20 mm can be found, respectively. The debonding loads could be predicted by considering the interfacial fracture energy and depended on the maximum aggregate size.
3. The testing results showed that the effective bond length increased with the aggregate size. Compared to the specimens with the aggregate size of 5-10 mm, a growth of 21.62% and 45.95% for the specimens with the size of 10-15 mm and 15-20 mm were observed, respectively.
4. Findings from the present tests showed that the specimens with the aggregate size of 10-15 mm and 15-20 mm experienced significant decrease in the peak shear stress up to 18.46% and 33.71% compared to the specimens with the size of 5-10 mm. The local slip at peak shear stress experienced significant increase with the aggregate size. An increase of 11.61% and 21.43% for the specimens with the aggregate size of 10-15 mm and 15-20 mm were found compared to the specimens with the aggregate size of 5-10 mm.
5. The proposed empirical model for the interfacial bond-slip relationship incorporating the effect of aggregate size can well predict the bond-slip behaviours.

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