BOND-SLIP MODEL FOR INTERFACES BETWEEN EXTERNALLY BONDED FRP REINFORCEMENT AND CONCRETE AT ELEVATED TEMPERATURES

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Abstract
This paper presents a nonlinear bond-slip model for fiber reinforced polymer (FRP) plates/sheets externally bonded to concrete at elevated temperatures as an extension of the two-parameter bond-slip model previously proposed by Dai et al. [1] for ambient temperature responses. The two key parameters of the model, the interfacial fracture energy $G_f$ and the interfacial ductility index $B$, were determined from the existing shear test data of FRP-to-concrete bonded joints at elevated temperatures. During the interpretation of the data, the effects of temperature-induced thermal stresses and temperature-induced bond degradation are appropriately isolated and dealt with. It is shown that the interfacial fracture energy remains almost constant initially and then decreases as the temperature increases; the interfacial ductility index exhibits a similar trend. The proposed bond-slip model is shown to compare well with the test data upon which it is based, in terms of both the ultimate load and the strain distribution in the FRP, demonstrating that the model closely represents bond behavior of FRP-to-concrete interfaces at elevated temperatures.

Keywords: Fiber reinforced polymer (FRP); Concrete; Interface; Bond-slip model; Bond strength; Elevated temperature; Interfacial fracture energy.

1. Introduction
The poor fire resistance of externally-bonded fiber reinforced polymer (FRP) reinforcement in the forms of pre-fabricated FRP plates and wet-layup FRP sheets (both are referred to as FRP plates except when explicit differentiation between them becomes necessary) is a major limitation of the FRP strengthening technology in indoor building applications. Both the matrix material of FRP and the bonding adhesive used in FRP strengthening are organic polymers (normally epoxies) which soften quickly above their glass transition temperature $T_g$ (°C) (generally in the range of 45 °C to 82 °C [2,3]). One solution to the problem is to cover the bonded FRP system with a fire protection layer of sufficient thickness to keep the temperature in the FRP below its glass transition temperature during a fire [4]. However, this approach is impractical or unattractive as the fire protection layer can become excessively thick and/or costly (e.g. 70 mm thick calcium silicate board claddings for a two-hour fire
A more practical solution is to adopt a relatively thin fire protection layer to achieve only partial insulation for the bonded FRP system which nevertheless ensures an adequate structural resistance during a fire. Depending on the situation, the consequence may be that the fire protection measure allows the original RC structure to completely or largely preserve its resistance during a fire but the structural resistance of the bonded FRP system is totally lost, or that the resistance offered by the bonded FRP system is partially retained during a fire.

To explore the benefits of different fire protection strategies and to develop corresponding design procedures, an accurate predictive capability for the behavior of FRP-strengthened RC structures with fire protection of various levels is needed. For the development of such a capability, a reliable bond-slip model for FRP-to-concrete interfaces exposed to elevated temperatures is essential. This paper presents a study aimed at the development of the firstbond-slip model for FRP-to-concrete interfaces at elevated temperatures.

2. Formulation of the bond-slip model

2.1 General

A bond-slip model for FRP-to-concrete interfaces can be developed on the basis of either local debonding mechanics (e.g. Ref. [6]) or global load-displacement responses of FRP-to-concrete bonded joints (e.g. Ref. [1]). Due to the lack of understanding of detailed local material degradation of FRP-to-concrete interfaces at elevated temperatures, the latter approach has been adopted in the present study. Indeed the proposed model represents an extension of the two-parameter bond-slip model developed by Dai et al. [1] for FRP-to-concrete interfaces at ambient temperature. Dai et al.’s approach has the advantage of simplicity as only two parameters need to be determined from test data. Another justification for using Dai et al.’s approach is that the softening of adhesive at elevated temperatures can be implicitly reflected in these two parameters. Although the use of only two parameters imposes some unnecessary constraint on the shape of the bond-slip curve, the bond-slip model proposed in this paper represents a good first attempt within the context of available information for FRP-to-concrete interfaces at elevated temperatures.

2.2 Theoretical background

The single-lap (or double-lap) shear test of bonded joints is a popular method for studying the bond characteristics of FRP-to-concrete interfaces (Fig.1). To obtain local bond-slip (τ-δ) curves from such a shear test, a series of strain gauges need to be installed on the surface of the bonded FRP plate. Bond-slip curves obtained directly from such strain measurements show considerable scatters due to the local bending of the FRP plate and the heterogeneity of the cracked concrete. To overcome these problems, Dai et al. [1] proposed a simple but rigorous method to derive the local bond-slip relationship from the test data.

![Figure 1. Schematic diagram of a single-lap shear test.](image)
At any location of the FRP-to-concrete interface in such a shear test, the relationship between the FRP strain and the interfacial slip can be described as

\[ \varepsilon(x) = f(\delta(x)) \]  

(1)

By assuming that the \( \tau-\delta \) relationship is unique along the FRP-to-concrete interface, Eq.1 becomes independent of location for a sufficiently long bond length [1,7]. Therefore, Eq.1 can be simply obtained from the pull force (from which the strain in the FRP at the loaded end can be found) and the relative interfacial slip between the FRP and the concrete at the loaded end. It should be noted that when the interface is subjected to combined mechanical and thermal loadings, the strain in the FRP plate at the loaded end consists of both load-induced and thermally induced components as discussed later in the paper.

From the interpretation of extensive test results, the following exponential expression has been found to represent \( f(\delta(x)) \) well [1]:

\[ \varepsilon(x) = A(1 - e^{-B\delta(x)}) \]  

(2)

where \( A \) and \( B \) are two parameters to be determined from regression analysis of test results. The physical meaning of \( A \) is the maximum strain reached in the FRP plate of a bonded joint when the bond length is longer than the effective bond length [8]. The parameter \( B \) can be regarded as a ductility index that determines the shape of the bond-slip relationship. A smaller value of \( B \) corresponds to a lower initial interfacial stiffness and a more gradual descending branch.

From Eq. 2, it is not difficult to obtain the following bond-slip relationship for an FRP-to-concrete interface [1]:

\[ \tau(x) = 0.5GB(\varepsilon^{-B\delta(x)} - \varepsilon^{-2B\delta(x)}) \]  

(3)

where \( G \) is the interfacial fracture energy (i.e. the area underneath the \( \tau-\delta \) relationship). The maximum strain reached in the FRP plate can be related to the interfacial fracture energy as follows:

\[ A = \frac{2G}{\sqrt{E_p\varepsilon_c}}(1 + \alpha) \]  

(4)

where \( \alpha = \frac{\varepsilon_p\varepsilon_c}{E_p\varepsilon_c} \); \( \varepsilon_p \) and \( \varepsilon_c \) are the width and thickness of the FRP plate; \( h_p \) and \( h_c \) are the width and thickness of the concrete prism; and \( E_p \) and \( E_c \) are the elastic moduli of the FRP plate and the concrete, respectively.

With Eqs 3 and 4, Dai et al. [7] derived an explicit expression for the slip distribution of an FRP-to-concrete interface with a sufficiently long bond length under mechanical loadings. Gao et al. [9] extended the expression to the following equation for an FRP-to-concrete interface subjected to combined thermal and mechanical loadings:

\[ \delta(x) = \frac{1}{b} \ln \left[ e^{\theta(\alpha_1 + \alpha_2 + L)} + 1 \right] \]  

(5)

where \( c_2 \) is a constant given by

\[ c_2 = \frac{1}{b} \ln \left[ \frac{\varepsilon_p^{(1+\alpha_1)}\varepsilon_c^{(1+\alpha_2)} + \varepsilon_c}{1 - \varepsilon_p^{(1+\alpha_1)}\varepsilon_c^{(1+\alpha_2)}} \right] - AL \]  

(6)

where \( P \) is the pull force acting on the FRP plate at the loaded end; \( L \) is the bond length; \( T \) is the temperature variation (a positive value means a temperature increase); \( \alpha_p \) and \( \alpha_c \) are the
thermal expansion coefficients of FRP and concrete, respectively. Once $\gamma(x)$ is known, the strain distribution, $\varepsilon(x)$, over the FRP plate at a given pull force can also be obtained as

$$\varepsilon(x) = \frac{A}{1+\thetaA(x-x)}$$

(7)

where $P_{UT}$ is the ultimate pull force (i.e. ultimate load) of an FRP-to-concrete bonded joint subjected to combined mechanical and thermal loadings and can be related to the interfacial fracture energy and the temperature variation as follows:

$$P_{UT} = \frac{2G_f \frac{E_p b_p}{(1+\nu)h_p}}{(1+\nu)^2} \left(\alpha_p - \alpha_c\right) \Delta T$$

(8)

When $\Delta T = 0$, Eq. 8 reduces to the familiar relationship between the ultimate load and the interfacial fracture energy of an FRP-to-concrete bonded joint at ambient temperature.

The relationship between the pull force and the interfacial slip at the loaded end of an FRP-to-concrete bonded joint with a sufficiently long bond length can be also obtained as follows [9]:

$$P = \frac{E_p b_p}{(1+\nu)} \left[A(1 - e^{-\eta \Delta}) - (\alpha_p - \alpha_c) \Delta T\right]$$

(9)

where $\Delta$ is the interfacial slip at the loaded end of the FRP plate.

2.3 Determination of $G_f$ and B

With the above theoretical background, a bond-slip curve of the form depicted by Eq. 3 with $G_f$ and $B$ as the key parameters can be determined from shear test results without difficulty for an FRP-to-concrete interface subjected to combined mechanical and thermal loadings. For a single-lap shear test or one of the four interfaces in a double-lap shear test (the latter has been widely used to evaluate the bond behavior of FRP-to-concrete interfaces at elevated temperatures [10-14]), the interfacial fracture energy $G_f$ can calculated from the ultimate load and Eq. 8 as follows provided that the bond length is longer than the effective bond length:

$$G_f(T) = (1 + \alpha) \frac{(P_{UT} - \Delta P)^2}{2E_p b_p}$$

(10)

where $\Delta P = -\frac{E_p b_p}{(1+\nu)} \left(\alpha_p - \alpha_c\right) \Delta T$, which is induced by thermal incompatibility between FRP and concrete. This incompatibility has a positive effect on the ultimate load when the temperature increases (and vice versa) provided that the bonding material has experienced no degradation. In deducing the temperature-dependent interfacial fracture energy from the ultimate load, $\Delta P$ needs to be isolated.

It should also be noted that the elastic modulus $E_p$ of the FRP in Eq.10 may also change with temperature. Bisby et al. [4] proposed a sigmoid function for the strength and stiffness degradation of pre-fabricated FRP bars/plates at elevated temperatures (Fig. 2a). The test data from Zhou [17] and Wang et al. [18] for FRP bars are also shown in the figure for comparison. It is clearly seen that the stiffness degradation of pre-fabricated FRP bars/plates and that of FRP sheets formed in a wet lay-up process [16,17] are different. To consider this difference, Dai et al. [19] proposed a modified model for the stiffness degradation of FRP sheets formed in a wet lay-up process by introducing the glass transition temperature $T_g$ into Bisby et al.'s model [4] as follows:
where $E_{p0}$ and $E_{pT}$ are the elastic modulus of FRP at ambient temperature and that at an elevated temperature $T$ (°C), respectively; and $a_1 = 0.729$, $a_2 = 9.856$ and $a_3 = 0.607$ are empirical factors derived from multivariable least-square regression analysis of existing test data (Fig. 2b). For pre-fabricated FRP plates at elevated temperatures, Bisby et al.’s model [4] developed for FRP bars is directly used in his study to describe the elastic modulus degradation due to the lack of test data on $T_G$ as well as the elastic moduli of FRP plates at elevated temperatures.

Once the value of $G_f$ is known, the value of $B$ can be obtained from least-square regression analysis of the relationship between the pull force and the local slip at the loaded end (i.e. Eq. 9) obtained from a shear test. Unfortunately, in all existing tests, only Klamer [12] reported the global pull force-displacement curves at elevated temperatures. For other tests, such relationships are not available. Instead, the strain distributions over the FRP plate at different pull force levels have often been reported. In the present study, the global load-displacement curve, if available, was used as the first choice to determine the value of $B$ for each FRP-to-concrete bonded joint. If it is not available, Eq. 7, which represents the theoretical strain distributions over the FRP plate at different pull forces, was then used for comparison with test results to calibrate the value of $B$. Preference was given to global load-displacement curves over measured strain values in the FRP because the latter are much more susceptible than the former to local effects such as the local bending of the FRP plate and the non-uniform distributions of coarse aggregate and cracks in the substrate concrete.

### 2.4 Expressions for $G_f$ and $B$

The data of a total of 79 FRP-to-concrete bonded joints tested at ambient and elevated temperatures were assembled from the published literature [10-14] to examine the dependence of $G_f$ and $B$ on temperature following the approach explained earlier. All these results were obtained from double-lap shear tests with a sufficiently long bond length. Details of the tests can be found in [19]. Through a careful analysis of the test data, a temperature-dependent bond-slip model in the form of Eq. 3 can be formulated with the following expressions for $G_f$ and $B$:

$$\frac{G_f(T)}{G_f(0)} = \frac{1}{2} \times \tanh \left( -b_2 \times \left( \frac{T}{T_0} - b_3 \right) \right) + \frac{1}{2} \quad (12)$$
\[
\frac{B(t)}{E_0} = \frac{(1-c_1)}{2} \times \tanh \left( -c_2 \times \left( \frac{T}{T_g} - c_3 \right) \right) + \frac{(1+c_2)}{2} \times \tanh \left( \frac{T}{T_g} - c_3 \right) + c_4
\]

where \( B_0 \) (mm) and \( G_{f0} \) (N/mm) are respectively the interfacial ductility index and the interfacial fracture energy at ambient temperature; and \( b_2 = 3.206, b_3 = 1.313, c_1 = 0.485, c_2 = 14.053 \) and \( c_4 = 0.877 \) are constants determined from least-square regression analysis of the test data. The values of \( B_0 \) and \( G_{f0} \) may vary within a wide range depending on the properties of concrete and adhesive mixture [1]. For use in the proposed bond-slip model, a small number of shear tests can be conducted to determine the two parameters for a specific type of FRP sheet/plate (with a given adhesive). Alternatively, if a conventional adhesive is used, the interfacial fracture energy can be estimated from Lu et al.'s model [6], and a value of 10.4 may be used for \( B_0 \) [7]. Figs. 3a and 3b present the predicted bond-slip curves at ambient and elevated temperatures for Blontrock’s test [10] and Klamer’s test [12] respectively. It is clearly seen that as the temperature increases, the initial slope of the bond-slip curve decreases and the descending branch becomes more gradual. Of course, the area beneath the bond-slip curve shrinks, indicating a reduction in the interfacial fracture energy due to elevated temperature exposure.

3. Validation of the proposed bond-slip model

Figure 4a presents a comparison between the ultimate load \( P_{u,T,pred} \) predicted using the proposed bond-slip model and the test ultimate load \( P_{u,T,exp} \) for all the 79 specimens of the present database. The average of the predicted-to-test load ratios and the coefficient of variation are 1.006 and 20.23%, respectively. This comparison indicates that the proposed bond-slip model leads to accurate predictions for the ultimate loads of FRP-to-concrete bonded joints at elevated temperatures. While this close agreement can be expected because the same test data were used to deduce the interfacial fracture energy \( G_f \), it at least demonstrates that the complex relationship between \( G_f \), the temperature and the glass transition temperature of the bonding adhesive are well captured by Eq. 12.

With the proposed bond-slip model and Eq. 7, the strain distributions over the FRP plate at different load levels can be predicted. Taking specimen Zijde-55 tested by Blontrock [10] as examples, Fig. 4b shows comparisons between the predicted and experimental strain distributions over the FRP plate. The bond length of the specimen was 300 mm, which is longer than the effective bond length, and the test temperature was 55 °C. In Fig. 4b, two sets of predictions are provided: (a) predictions using Eq. 7 and \( B = 597 \) mm, which was regressed...
Figure 4 Comparisons between predictions and experimental data.

4. Conclusions

Based upon a careful analysis of the existing data of 79 shear tests on FRP-to-concrete bonded joints at temperatures ranging from 4 °C to 180 °C, a nonlinear temperature-dependent bond-slip model for FRP-to-concrete interfaces has been proposed. This model is an extension of the two-parameter bond-slip model previously proposed by Dai et al. [1] for the ambient temperature condition. An important aspect of the approach employed in this study is that the effect of thermal incompatibility between FRP and concrete is properly isolated and dealt with in the interpretation of test data. The proposed bond-slip model has been shown to closely represent the test data in terms of both the ultimate load of the bonded joint and the strain distribution in the FRP.

5. Acknowledgements

The authors are grateful for the financial support received from the Research Grants Council of the Hong Kong SAR (Project No: PolyU 516509) and for a PhD studentship awarded to the first author by The Hong Kong Polytechnic University.

6. REFERENCES


