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IC debonding failure in RC beams strengthened with FRP: Strain-based versus stress increment-based models

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ABSTRACT

Since the advent of the use of fibre-reinforced polymers (FRP) to strengthen reinforced concrete (RC) and its employment in civil engineering in the early nineteen nineties, much of the research has concerned the enhancement of bending capacity. Failure modes are well understood and can be predicted with some accuracy. The models adopted by the most highly reputed international standards for the specific case of inter-crack (IC) debonding are based on limiting the strain on the laminate. These methods, an offshoot of fracture mechanics analysis, are calibrated with the results of point load beam tests. This paper proposes a new interfacial fracture energy model by characterising the FRP-concrete interface with beam-type tests. The existing fracture energy models, as well as the proposed model, were applied to an existing stress increment-based formulation, in which no calibration was conducted with experimental results and yet proved to predict inter-crack debonding failure moderately better than the models in use. The performance of each IC debonding model is assessed through an exhaustive statistical analysis with experimental data of point load RC beams tests gathered from literature. The behaviour of the models in uniformly loaded RC beams is discussed and the implications of each approach are drawn.

1. Introduction

The vast majority of IC debonding models are based on fracture mechanics analysis of pure shear (mode II). In Taljsten's [1] early proposals based on linear elastic fracture mechanics and assuming no deformation in the concrete substrate, the maximum transferable force is given by Eq. (1).

$$F_{\max} = b_p\sqrt{2G_f E_p t_p} \tag{1}$$

where G_f is the interfacial fracture energy and b_p, t_p and E_p are width, thickness and modulus of elasticity of the FRP material respectively. Shear stress distribution in debonding has been studied by a number of researchers. Teng et al. [2] analysis is particularly significant, for it envisages tensile stress acting on both sides of the laminate between two adjacent cracks, which can be used as a proxy for crack-induced debonding failure. In that analysis, expressions for interfacial shear stress, tensile stress on the FRP laminate and interfacial slip are defined for each stage of debonding. Shear stress distribution can be readily calculated by numerical analysis, however, where the FRP-concrete interface is modeled using spring elements whose mechanical behaviour is defined by the bond-slip relationship [3]. Teng et al. [2] analyses assume a linear bond-slip relationship, while the numerical solution accommodates analysis of different shapes of the descending branch of this function. Chen et al. [4] conducted a simpler analysis that addresses the descending branch of the bond-slip relationship only. That simplification gives an explicit expression for ultimate load: by assuming no deformation in the concrete substrate the expression adopts the form of Eq. (1) multiplied by the term which accounts for the effect of bonded length and load ratio of debonding:

$$\beta_L = \begin{cases} \frac{1}{\sqrt{1-\upsilon^2}} & \text{if } L_b \geqslant L_s \\ \frac{\sin(\lambda L_b)}{1-\upsilon\cos(\lambda L_b)} & \text{if } L_b < L_s \end{cases} \tag{2}$$

where

$$L_s = \frac{\arccos\upsilon}{\lambda} \quad \text{and} \quad \lambda = \frac{\tau_{\max}}{\sqrt{2G_f E_p t_p}} \tag{3}$$

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In the above equations, υ is the ratio between the lowest and highest FRP force, L_b is the bonded length (i.e. crack spacing), L_s is the characteristic softening length and $\tau_{\max}$ is the maximum shear stress.

Comparisons of IC debonding models have shown that strain limitation-based models have greater prediction power [5]. In this paper, the existing interfacial fracture energy models, as well as the proposed model, were applied to the Chen et al. [4] formulation. In this formulation no calibration was conducted with experimental results; however, it proved to predict IC debonding failure moderately better than the models in use.

2. IC debonding models

Three major approaches to predicting IC debonding can be defined: limitation of strain in the FRP laminate at the cross-section where the bending moment is highest; mean bond stress (expressed as the difference in the tensile forces on the FRP laminate divided by the bond area); and the allowable increment in stress on the FRP composite.

2.1. Strain limitation approach

Earlier studies suggested limiting the strain in the FRP laminate to a value ranging from 0.0065 to 0.0085 (*fib* [6]). However, and since IC debonding depends on a number of variables, a fixed value could lead to uneconomical solutions. Later methods were based either on calibrating Eq. (1) (e.g. ACI-440.2R-08 [7], Eq. (4)) or developing empirical models, as proposed by Said and Wu [5] (Eq. (5)).

$$\varepsilon_{pd} = 0.41\sqrt{\frac{f_c}{t_p E_p}} \leqslant 0.9\varepsilon_{pu} \tag{4}$$

$$\varepsilon_{pd} = 0.23\frac{f_c^{0.2}}{(E_p t_p)^{0.35}} \tag{5}$$

In the above equations, f_c is the concrete compressive strength, which is a significant variable of the interfacial fracture energy.

2.2. Mean bond stress approach

This approach is described in *fib* Bulletin 14 [6] as an alternative method for verifying tangential bond stress, whose mean value, τ_{bd}, is limited to $1.8f_{ctk}/\gamma_c$. However, inasmuch as this model yields highly scattered results, and is overly optimistic in most cases [5,8], it has not been included in the present comparative study.

2.3. Stress increment limitation approach

Both Japanese standard [9] and *fib* Bulletin 14 [6] address this approach. In the Japanese code, the limitation is defined by Eq. (1), expressed as stress on the laminate (Eq. (6)), using a design length of 150–250 mm. In the absence of tests, the standard suggests using $G_f = 0.5$ N/mm.

$$\Delta\sigma_{pd} = \sqrt{\frac{2G_f E_p}{t_p}} \tag{6}$$

Wu and Niu [10] proposed finding the rise in stress in a given length as the greater of the double of the effective bond length and the distance between the critical section and the end of the yield region at the debonding load, L_y. Based on the results from double-face shear type tests conducted by Nakaba et al. [11], the aforementioned authors suggested calculating the interfacial fracture energy and the effective bond length with Eqs. (7) and (8), respectively.

$$G_f = 0.644 f_c^{0.19} \tag{7}$$

$$L_{eb} = \frac{0.649\sqrt{E_p t_p}}{f_c^{0.095}} \tag{8}$$

In the more complex procedure described in *fib* Bulletin 14 [6], the allowable stress increment depends on the lowest value in the area studied. As might be intuitively expected, this procedure calls for iteration and is consequently too complex for practical applications. It consists of three main stages. In the first one, the least favourable crack spacing is calculated from the following expression, assuming mean bond stress for both the internal steel reinforcement and the external strengthening:

$$s_{rm} = 2\frac{M_{cr}}{z_m}\left(\frac{1}{\sum \tau_{pm} b_p + \sum \tau_{sm} d_s \pi}\right) \tag{9}$$

where d_s is the diameter of the steel bars. M_{cr} is the cracking moment, z_m is the mean lever arm of internal forces, τ_{sm} and τ_{pm} are the mean bond stress of the internal steel reinforcement and the external strengthening respectively. These parameters are calculated as follows:

$$M_{cr} = 2f_{ctk,0.95} bh^2/6 \tag{10}$$

$$z_m = 0.85\frac{hE_pA_p + dE_sA_s}{E_pA_p + E_sA_s} \tag{11}$$

$$\tau_{sm} = 1.85 f_{ctm} \tag{12}$$

$$\tau_{pm} = 0.44 f_{ctm} \tag{13}$$

where $f_{ctk,0.95}$ is the upper bound characteristic tensile strength of concrete, f_{ctm} is the mean value of the tensile concrete strength, A_p is the cross sectional area of the FRP reinforcement, A_s is the area of the longitudinal steel reinforcement, E_s is the modulus of elasticity of the steel reinforcement and d, b and h are the effective depth, the width and the total depth of the beam respectively.

In the second stage, the stress on the laminate at each crack is found via force equilibrium and strain compatibility, which entails applying the shift rule to the law of moments as per EC-2 [12]. The maximum allowable stress increment is calculated as shown below:

If $\sigma_{p1} \leqslant \sigma_p^{(B)}$

$$\Delta\sigma_{pd} = \Delta\sigma_{p,\max}^{(A)} - \frac{\Delta\sigma_{p,\max}^{(A)} - \Delta\sigma_{p,\max}^{(B)}}{\sigma_p^{(B)}}\sigma_{p1} \tag{14}$$

If $\sigma_{p1} > \sigma_p^{(B)}$

$$\Delta\sigma_{pd} = \min\left[1/\gamma_c\left(\sqrt{c_1^2 E_p\sqrt{f_{ck} f_{ctm}}/t_p + (\sigma_{p1})^2} - \sigma_{p1}\right), (f_{pu} - \sigma_{p1})\right] \tag{15}$$

where

$$\Delta\sigma_{p,\max}^{(A)} = (c_1/\gamma_c)\sqrt{E_p\sqrt{f_{ck} f_{ctm}}/t_p} \tag{16}$$

$$\sigma_p^{(B)} = c_3 E_p/s_{rm} - c_4\sqrt{f_{ck} f_{ctm}}\, s_{rm}/4t_p \tag{17}$$

$$\sigma_{p,\max}^{(B)} = 1/\gamma_c\left(\sqrt{c_1^2 E_p\sqrt{f_{ck} f_{ctm}}/t_p + (\sigma_p^{(B)})^2} - \sigma_p^{(B)}\right) \tag{18}$$

f_{ck} is the characteristic value of the concrete compressive strength, γ_c is the material safety factor for the concrete and σ_{p1} is the lowest stress in the area studied. The values of constants $c1$, $c2$, $c3$ and $c4$ are 0.23, 1.44, 0.185 and 0.285, respectively. These constants are related to the stress-slip relationship as follows: