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DESIGN PROPOSALS FOR THE DEBONDING STRENGTHS OF FRP STRENGTHENED RC BEAMS IN THE CHINESE DESIGN CODE

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ABSTRACT

Debonding failures are very common in FRP strengthened RC structures so they must be carefully considered in design. In the last few years, significant new understandings of the debonding behaviour of both flexurally and shear strengthened RC beams have been achieved based on recent research on the behaviour of FRP-to-concrete interface. These new understandings are reflected in the national design standard of China, Standard for FRP in Civil Engineering, which is being drafted. This paper summarises relevant specifications adopted for such debonding failures in the new Chinese standard.

KEYWORDS

FRP, strengthening, design specifications, debonding, flexural, shear

INTRODUCTION

The technique of externally bonding fibre reinforced polymer (FRP) composite plates or sheets (referred as either FRP sheets or FRP plates interchangeably hereafter for brevity) to reinforced concrete (RC) structures was introduced into China in 1997. After extensive research and promotion since then, it has now become a major method for retrofitting concrete structures. At present, over 600,000m^2 FRP sheets are used to retrofit or repair concrete structures every year in China. This popularity of FRP in China means that there is a strong demand for design standards and specifications. Consequently, the first specification for FRP in Civil Engineering in China, “Technical specification for strengthening concrete structure with carbon fibre reinforced polymer laminate CECS-146” (CECS-146 2003) (referred as the specification hereafter for brevity), was published in 2003. A national standard, “Standard for FRP in Civil Engineering” (referred as the standard hereafter for brevity), is currently being developed.

When an RC structure is strengthened with externally bonded FRP, the bond between FRP and concrete plays a crucial role in guaranteeing the effectiveness of the strengthening. Because of the high strength of FRP materials, most failures of FRP strengthened RC members are caused by debonding along FRP-to-concrete interfaces. herefore, appropriate considerations must be given to debonding failures in design. This paper presents the design methods proposed in the specification and the standard for debonding failures in flexurally and/or shear strengthened RC beams.

The debonding failures concerned in this paper are limited to cases where the FRP sheets are properly installed with appropriate adhesives by qualified personnel following a proper construction procedure. In such cases, debonding failures usually occur in the concrete about 2~10mm away from the concrete-adhesive interface. It is assumed that any debonding in the adhesive or delamination of FRP is caused by poor construction or substandard materials. Such failures should be prevented by careful selection of materials, quality assurance of supplied materials and by following appropriate construction and inspection procedures. These details are not discussed in this paper.

DEBONDING FAILURES IN FLEXURALLY STRENGTHENED MEMBERS

A large number of experimental studies (e.g. Teng *et al.* 2002a, 2003a, b, Lu 2004) have shown that, without any additional anchorage, there are mainly three debonding failure modes in RC beams strengthened with a tension face FRP sheet (Figure 1).

1) Plate end debonding/concrete cover separation;
2) Critical diagonal crack debonding (CDC debonding);
3) Intermediate crack induced debonding (IC debonding).

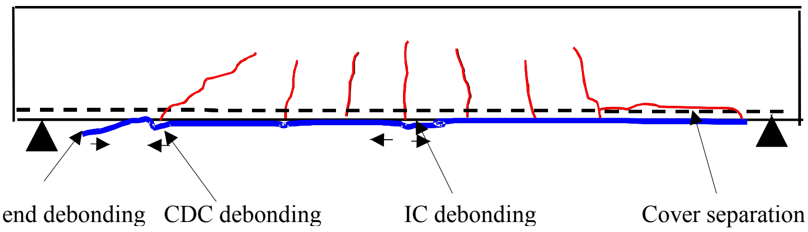


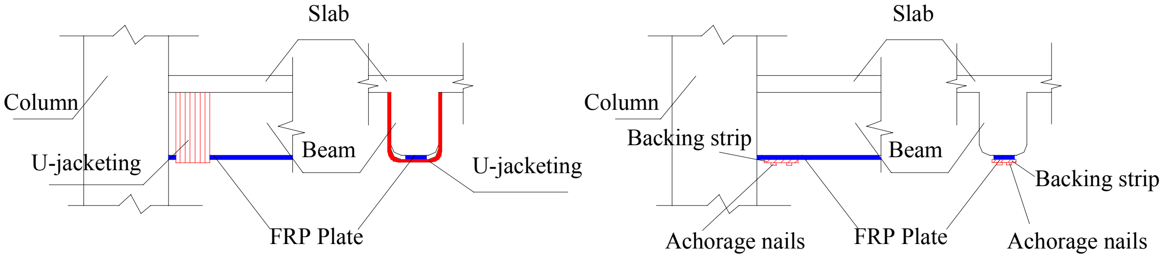
Figure 1. Debonding failure modes in flexurally-strengthened RC beams

Plate End Debonding/Concrete cover separation

FRP plate end debonding or concrete cover separation is believed to be caused by the significant stress concentration at the FRP plate end arising from geometrical and flexural stiffness discontinuities. This failure mode has received extensive attentions in early studies on FRP strengthening of RC structures. Linear elastic analysis indicates that very large normal and shear stresses exist in the adhesive layer at the plate end (Teng *et al.* 2002a, 2002b). Many factors including the elastic modulus and the thickness of the adhesive layer affect the values of these stresses. It shall be noted that these large stresses are present only in a small region: they are reduced to very small values several times of the thickness of FRP plate away from the plate end. Because the thickness of the FRP plate is only a few millimetres in most cases, the actual size of the stress concentration region is very small.

Since the debonding always occurs within the concrete, the actual stress distributions at the FRP-to-concrete interface are much more complicated than those from a linear elastic analysis due to concrete cracking. This led to the development of several design proposals considering the nonlinear interfacial behaviour. However, there are still large discrepancies between all strength models based on either linear elastic or nonlinear interfacial stress analyses and test results (Smith and Teng 2002a, b).

Further research has shown that the plate end debonding/concrete cover separation can be easily prevented by using additional anchors such as FRP U-jackets or nails at the FRP plate ends (Figure 2). The installation of such anchors at the plate ends is very convenient in practice. Therefore, both the specification and the standard proposed the following clause to avoid plate end debonding/concrete cover separation: "*The tension face FRP plates/sheets should be extended to the supports. FRP U-jackets should be installed at the ends of FRP plates/sheets. The width and thickness of FRP U-jackets should not be less than half of the width and thickness of the tension face FRP plates/sheets.*"



(a) FRP U-jacketing (b) Nail anchors

Figure 2 Additional anchors for preventing plate end debonding

If there are difficulties in installing such plate end anchors, it was recommended that the conservative model proposed by Smith and Teng (2002b) be used to calculate the debonding strength. But the strength of FRP may not be fully used in such cases.

Critical Diagonal Crack Debonding

The opening-up of a diagonal shear crack induces not only interfacial shear stresses but also interfacial normal stresses at the FRP-to-concrete interface due to the relative sliding displacement between the two sides of the shear crack of a concrete beam. The development of the shear crack leads to not only the shear failure of the beam, but also debonding of the FRP from the concrete starting from the shear crack. Such debonding failure is termed the Critical Diagonal Crack (CDC) debonding (Mohamed Ali *et al.* 2001, 2002, Oehlers *et al.* 2003). A CDC debonding failure is very brittle.

The main cause of CDC debonding failure is the low shear capacity of the beam. An effective method for preventing CDC debonding is thus to avoid shear failure of a beam by increasing its shear capacity. RC beams are usually designed following the principle of "strong shear weak bending" to avoid the brittle shear failure. This principle also applies to FRP strengthened concrete beams, i.e. the shear capacity of a strengthened beam should be larger than its flexural capacity after flexural strengthening. Both the specification and the standard adopt this principle to avoid the CDC debonding. Furthermore, additional FRP U-jackets are also required to ensure the shear capacity of the flexurally strengthened beam even if its shear strength is adequate and to increase the ductility in an intermediate crack induced debonding failure (IC debonding). Further details are given in the following sections.

Intermediate Crack Induced Debonding

For an FRP strengthened RC beam designed to satisfy the principle of "strong shear weak bending" and various detailing requirements, flexural cracks will inevitably occur under service load. The initiation and development of flexural cracks result in large interfacial stresses at the FRP-to-concrete interface at both sides of a flexural crack which may lead to interfacial debonding failure. Such debonding failure is referred as Intermediate Crack induced debonding, or IC debonding (Teng et al. 2003b).

An IC debonding is caused by the widening of a flexural crack. The contribution of un-prestressed FRP to the flexural strength takes place mainly after the yielding of the flexural steel reinforcement which leads to rapid propagation of flexural cracks and large interfacial slips between the FRP and the concrete on both sides of the flexural crack. No efficient method is available yet to avoid IC debonding failures. If the thickness of the FRP plate is significant, IC debonding cannot be avoided even when additional anchors such as U jacketing are used (e.g. Lu 2004, Teng *et al.* 2002a). Therefore, IC debonding should be considered as one of the controlling failure modes in the strengthening design of RC beams using tension face FRP sheets. The flexural strength should be calculated by considering the effective FRP tensile stress at IC debonding failure.

Based on recent research on the mechanics of IC debonding (Lu 2004, Lu *et al.* 2004), the effective FRP strain at IC debonding can be calculated as:

$$\varepsilon_{f,IC} = (0.492/\sqrt{E_f t_f} - 0.086/L_d)\tau_{\max} \tag{1a}$$

where

$$\tau_{\max} = 1.5\beta_w f_t \tag{1b}$$

$$\beta_w = \sqrt{(2.25 - b_f/b_c)/(1.25 + b_f/b_c)} \tag{1c}$$

in which E_f (MPa) is the elastic modulus of the FRP plate; t_f (mm) is the thickness of the FRP plate; L_d (mm) is the distance from the plate end to the section where the FRP plate is fully used (Figure 3); β_w is the FRP-to-concrete width ratio; b_f is the width of the FRP plate; b_c is the width of the concrete beam; and f_t (MPa) is the average tensile strength of the concrete.

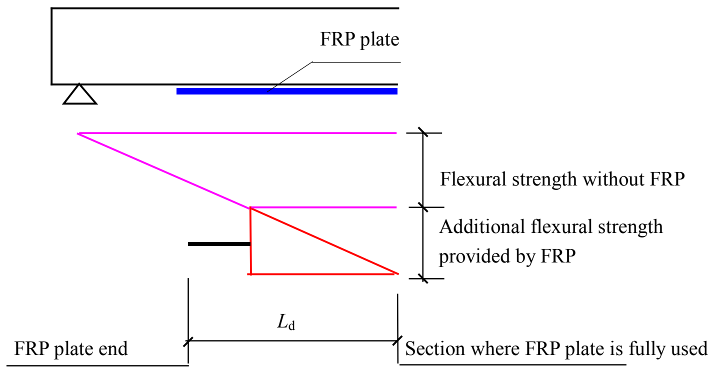


Figure 3 Definition of L_d

It may be noted that the concrete tensile strength in Eq. 1 is the average strength from tests. In the Chinese concrete design code, the design tensile strength is about half of the average strength. This can lead to over-conservative designs when the concrete design tensile strength is used. Consequently, the standard proposes some modifications to the coefficients in Eq. 1 to balance safety and economy.

Furthermore, although FRP U-jackets cannot completely prevent the occurrence of IC debonding, they may increase the ultimate failure load to certain extent. According to Zhuang (2005), properly installed FRP U-jackets can increase the FRP strain at an IC debonding failure by about 30%. Because of the limitation of available experience, the standard proposes that the contribution of U jackets to the IC debonding strength should not be considered if they do not meet the detailing specifications. However, any use of FRP U-jackets is beneficial because they can significantly improve the ductility in an IC debonding failure.

Finally, the standard adopts the following equation to calculate the FRP strain at IC debonding

$$\varepsilon_{fe,m2} = \lambda(1.0/\sqrt{E_f t_f} - 0.2/L_d)\beta_w f_{td} \tag{2}$$

where, f_{td} (MPa) is the design tensile strength of concrete; λ is a factor considering the anchorage of the tension face FRP plate. λ =1.0 in all cases, but = 1.3 (which may be increased to 1.5 if the designer has sufficient experience) when FRP U-jackets are used and meet the following detailing requirements:

"FRP U-jackets are installed within the whole length of L_d as in Figure 3. The height of U-jackets is not smaller than half of the beam height. It is preferable that the U-jackets are extended to the full height of the beam or to the bottom of the slab. The width and thickness of U-jackets shall not be smaller than half of the width and thickness of the tension face FRP plate respectively" (Figure 4).

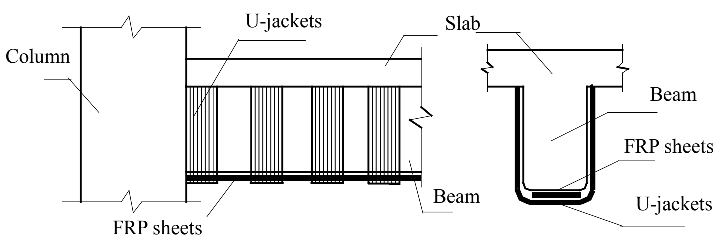


Figure 4 FRP U-jackets for flexural strengthening

At IC debonding failure, the extreme fibre of the compressive concrete may not reach its ultimate strain ε_{cu}. The concrete strength in the compression zone cannot be fully developed in this case and the compression force resisted by the concrete will be overestimated if the compressive stress block used in the design of normal RC beams is adopted. Based on some theoretical analyses, the standard adopts a factor ω to consider the reduction of the compressive concrete strength in such a case. Consequently, the bending moment capacity at IC debonding failure for an RC beam with tension face FRP plates is calculated according to the following equations:

$$M \le \omega f_c bx\left(h_0 - \frac{x}{2}\right) + \sigma_{f,md} A_f (h - h_0) \tag{3a}$$

$$\omega f_c bx = f_y A_s + \sigma_{f,md} A_f \tag{3b}$$

$$\sigma_{f,md} = \min\{f_{fd}, E_f \varepsilon_{fe,m1}, E_f \varepsilon_{fe,m2}\} \tag{3c}$$

$$\omega = 0.7 + 0.3 \frac{\varepsilon_{fe,m2}}{\varepsilon_{fe,m1}} \tag{3d}$$

where, b is the width of a rectangular cross-section; h_0 is the effective height of the section which is the distance from the top of the beam to the centroid of the tensile reinforcements; x is the height of the rectangular concrete compressive stress block; A_s is the cross-sectional area of the tensile steel reinforcements; A_f is the cross-sectional area of the tension face FRP plate; f_c is the design compressive strength of concrete; f_y is the design strength of steel reinforcement; $\sigma_{f,md}$ is the design strength of FRP for the ultimate limit state; f_{fd} is the FRP design tensile strength; and $\varepsilon_{fe,m2}$ is the effective FRP strain at IC debonding failure which can be obtained from Eq. 2 and should be not be smaller than $0.5\varepsilon_{fe,m1}$ in which $\varepsilon_{fe,m1}$ is the FRP strain when the outmost concrete fibre reaches the crushing strain as given by

$$f_c bx = f_y A_s + E_f \varepsilon_{fe,m1} A_f \tag{4a}$$

$$x = \frac{0.8\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fe,m1}} h \tag{4b}$$

A comparison with 80 IC debonding test data (Figure 5) shows that the adopted design model is accurate and conservative.

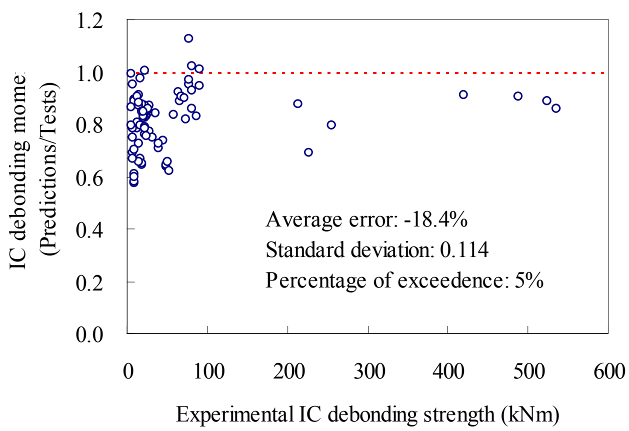
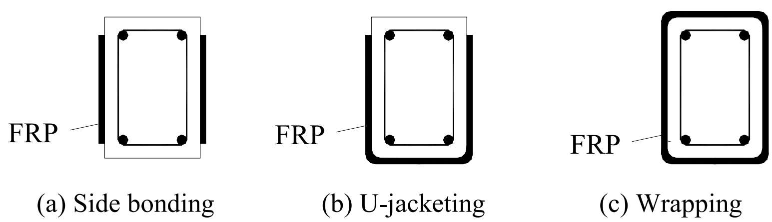


Figure 5. Adopted design model for IC debonding versus test data

DEBONDING FAILURE IN SHEAR STRENGTHENED RC BEAMS

As summarized by Chen and Teng (2003a), common methods for the shear strengthening of concrete beams include side bonding, U-jacketing and wrapping (Figure 6). U-jacketing and side bonding are more commonly used in practice because complete wrapping has difficulties such as the need to cut holes through concrete slabs.



(a) Side bonding (b) U-jacketing (c) Wrapping
Figure 6 Common methods for the shear strengthening of RC beams

Extensive research has shown that FRP fracture and FRP debonding are the two main failure modes in RC beams strengthened in shear (Chen and Teng 2003a, b). The FRP fibres intersecting a major shear crack are vulnerable to debonding failure when the crack widens. For almost all beams strengthened with side bonding and most beams strengthened with U jacketing, the shear capacity of the strengthened beam is controlled by debonding failure unless the ends of the FRP are properly anchored (Chen and Teng 2003a, b).

Most existing design models use the superposition principle to calculate the shear capacity of RC beams shear strengthened with FRP, i.e.

$$V_u = V_{RC} + V_{FRP} \tag{5}$$

in which V_{FRP} is the FRP contribution to the shear capacity and V_{RC} is the shear capacity of the original RC beam. It may be noted that the superposition principle is not precisely applicable here as V_{RC} and V_{FRP} may not reach their peak values simultaneously (Ye *et al.* 2002, Teng et al. 2002a) but it is still widely adopted for convenience and the same is adopted in the standard. The major task here is thus to determine the FRP contribution to the shear capacity V_{FRP}.

The value of V_{FRP} may be estimated from the summation of forces in the FRP stripes intersecting the critical shear crack at the ultimate limit state:

$$V_{FRP} = \sum \varepsilon_{fi} E_{fi} A_{fi} \sin\alpha = \varepsilon_{fe} E_f A_f \sin\alpha \qquad (6)$$

where ε_{fi} , E_{fi} and A_{fi} are the strain, elastic modulus and cross-sectional area of the i^{th} FRP strip; A_f is the total cross-sectional area of FRP intersecting the critical shear crack; α is the angle between the FRP fibre direction and the longitudinal axis of the beam; ε_{fe} is the average strain for FRP intersecting the shear crack at the ultimate limit state.

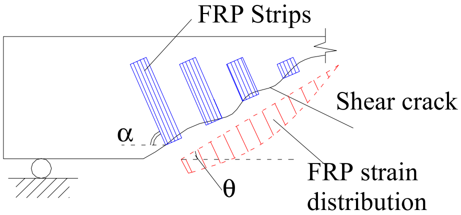


Figure 7 FRP Strain distribution along the critical shear crack

If the critical shear crack has an angle θ with the longitudinal axis of the beam, Eq. 6 can be rewritten as:

$$V_{FRP} = 2\varepsilon_{fe} E_f t_f w_f \frac{h_{fe}(\cot\theta + \cot\alpha)\sin\alpha}{s_f} \qquad (7)$$

where the factor of 2 applies when FRP is bonded on both sides of the beam; h_{fe} is the effective bond height of FRP as defined in Figure 8; and w_f and s_f are the width and the center-to-center spacing of FRP stripes respectively.

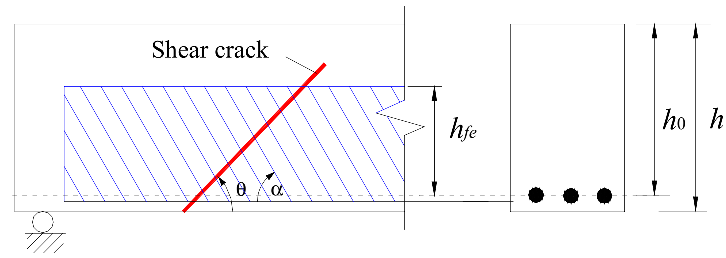


Figure 8 Effective bond height h_{fe}

Experimental and analytical studies have shown that the average strain of FRP intersecting the shear crack ε_{fe} is mainly controlled by the following factors: the strength of concrete, the FRP strengthening method, the bond length and stiffness of FRP plates, and the shape of shear cracks. Lu *et al.* (2004) recently investigated the average FRP strain ε_{fe} at debonding failure using a rigorous FRP-to-concrete interface bond-slip model (Lu *et al.* 2005a). They considered 4 typical shear crack shapes and found that Chen and Teng's (2003b) model developed based on their bond strength model (Chen and Teng 2001) is in good agreement with the numerical results (Lu *et al.* 2005b). Nevertheless, Lu (2004) proposed the following alternative but simpler model

$$\varepsilon_{fe} = K_v \varepsilon_{f,\inf} \qquad (8a)$$

where $\varepsilon_{f,\inf}$ is the FRP strain when the bond length is infinite and K_v is the FRP bond length effect factor that is expressed as:

$$K_v = \begin{cases} 0.77(1 - e^{-\lambda/0.79}) & \text{Side bonding} \\ 0.96(1 - e^{-\lambda/0.62}) & \text{U - jacketing} \end{cases} \qquad (8b)$$

in which the bond length ratio λ is the ratio of the FRP effective bond height h_{fe} to the FRP effective bond length L_e which is expressed as

$$\lambda = \frac{h_{fe}}{2L_e \sin\alpha} \qquad (8c)$$

$$L_e = 1.33 \frac{\sqrt{E_f t_f}}{f_t} \tag{8d}$$

The FRP strain $\varepsilon_{f,\text{inf}}$ for an infinite bond length is given as (Lu *et al.* 2005a):

$$\varepsilon_{f,\text{inf}} = \beta_w \sqrt{\frac{0.616\sqrt{f_t}}{E_f t_f}} \text{ , where } \quad \beta_w = \sqrt{\frac{2.25 - w_f / s_f}{1.25 + w_f / s_f}} \tag{8e}$$

During the process of developing the standard, the factors in the above equations were modified to consider the difference between the average and design concrete strengths whilst some simplifications were made. Assuming that the angle of the critical shear crack is 45°, the standard finally adopts the following design model:

$$V_{\text{FRP}} = \psi_v K_f \tau_{ave} w_f \frac{h_{fe}^2 (\sin\alpha + \cos\alpha)}{s_f} \tag{9a}$$

$$K_f = \phi \frac{\sin\alpha \sqrt{E_f t_f}}{\sin\alpha \sqrt{E_f t_f} + 0.3 h_{fe} f_t} \tag{9b}$$

$$\tau_{ave} = 1.2 \beta_w f_{td} \tag{9c}$$

$$\beta_w = \sqrt{(2.25 - w_f / s_f)/(1.25 + w_f / s_f)} \tag{9d}$$

where τ_{ave} is the average bond strength between FRP and concrete; and the shear strengthening method factor ϕ = 1.0 for side-bonding and 1.3 for U-jacketing.

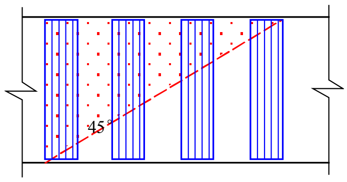
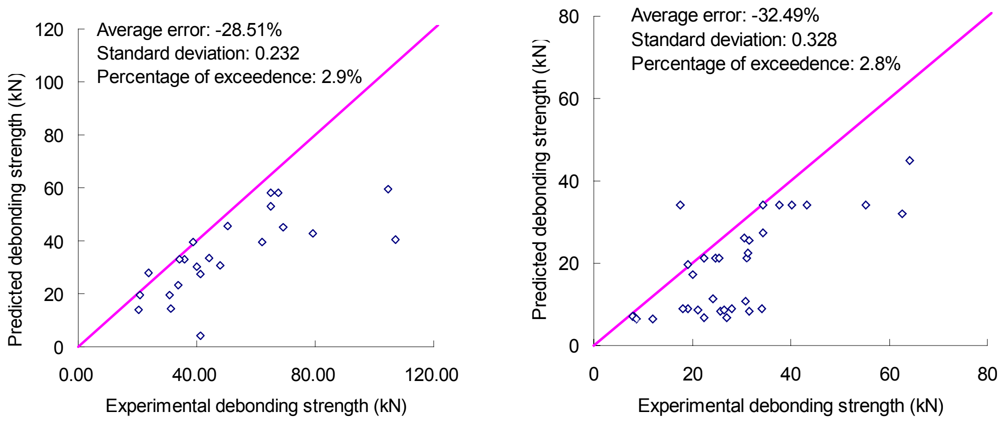


Figure 9 Effective FRP bond area

The proposed design equation (Eq. 9) for calculating V_{FRP} has been compared with 35 U-jacketing and 34 side-bonding test data. Figure 10 shows that the simple design model is in good agreement with test data and conservative.



(a) FRP U-jacketing (b) Side bonding

Figure 10 Debonding strengths of FRP shear strengthened RC beams: adopted model versus test data

CONCLUSIONS

This paper has presented a summary of some recent numerical and experimental research on FRP-to-concrete interfacial behaviour and debonding failures in FRP strengthened RC beams, and how some of these results have been modified for adoption into the Chinese Standard for FRP in Civil Engineering which is being developed. Comparisons of these simplified design models with test data have shown that the adopted design provisions are conservative and correlate well with the test data.

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