DEBONDING FAILURES OF RC BEAMS STRENGTHENED WITH EXTERNALLY BONDED FRP REINFORCEMENT: BEHAVIOUR AND MODELLING

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ABSTRACT

Both the flexural and shear strengths of reinforced concrete (RC) beams can be substantially enhanced using externally bonded fibre reinforced polymer (FRP) composites. Failures of such FRP-strengthened RC beams often occur by debonding of the FRP plate from the RC beam in a number of forms. Despite numerous theoretical and experimental studies on debonding failures of FRP-strengthened RC beams, considerable uncertainty still exists with the understanding of the failure mechanisms and with the accurate prediction of debonding failure loads. This paper provides a summary of the author’s understanding of the subject largely based on the research of the authors and their co-workers. The paper addresses the following three issues: (a) classification of debonding failure modes; (c) mechanisms and processes of debonding failures; and (e) theoretical models for debonding failures. The information presented in this paper may serve as a useful basis for the future development of design provisions in design codes and guidelines.

KEYWORDS

FRP, RC beams, strengthening, debonding, behaviour, modelling, design

INTRODUCTION

Strengthening reinforced concrete (RC) structures with externally bonded fibre reinforced polymer (FRP) composites has become a popular technique in recent years (e.g. Teng et al. 2002). The technique may be used to enhance the load-carrying capacities of RC beams, slabs and columns as well as the ductility of RC columns through lateral confinement. This paper is concerned with RC beams that are strengthened with externally bonded FRP reinforcement in the forms of sheets/strips/plates (all referred to as plates unless specific differentiation becomes necessary) in either flexure or shear.

FRP flexural strengthening of RC beams is commonly achieved by bonding an FRP plate to its tension face (Figure 1). Existing research has shown that such strengthened beams often fail by debonding of the FRP plate from the beam in one of several possible modes (e.g. Teng et al. 2000; 2002; Smith and Teng, 2002a; 2002b; Lu et al. 2007; Yao and Teng 2007). Despite the extensive existing research, there is still considerable uncertainty regarding many aspects of debonding failures, including the classification of debonding failure modes. The shear capacity of an RC beam can also be effectively enhanced using externally bonded FRP reinforcement in the form of complete wraps, U jackets or side strips (Figure 2). Debonding of FRP from concrete occurs in almost all RC beams shear-strengthened with FRP (Chen and Teng 2003a, 2003b; Cao et al. 2005) although in the case of complete wraps, debonding does not directly controls the ultimate load (Cao et al. 2005).

Figure 1 RC beam with an FRP plate bonded to its soffit

This paper provides a summary of the author’s understanding of the subject largely based on the research of the authors and their co-workers. The paper starts with a systematic classification of debonding failure modes,
followed by detailed descriptions of the mechanisms and processes of different debonding failure modes. The paper then presents theoretical strength models for debonding failures. For simplicity, all discussions in this paper are presented with explicit reference to a simply supported beam, but the design procedure can be easily applied to indeterminate beams.

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Figure 2 Shear strengthening schemes for RC beams using externally bonded FRP reinforcement

DEBONDING FAILURE MODES OF FLEXURALLY-STRENGTHENED RC BEAMS

Classification of Failure Modes

A number of distinct failure modes of RC beams bonded with an FRP soffit plate (i.e. FRP-plated RC beams) have been observed in numerous experimental studies (Teng et al. 2002; Buyukozturk et al. 2004; Oehlers and Seracino 2004; Yao and Teng 2007). A schematic representation of these failure modes is shown in Figures 3 and 4. Failure of an FRP-plated RC beam may be by the flexural failure of the critical section (Figure 3) or by debonding of the FRP plate from the RC beam (Figure 4). In the former type of failure, the composite action between the bonded plate and the RC beam is maintained up to failure, while the latter type of failure involves a loss of this composite action. Debonding failures generally occur in the concrete, which is also assumed in the strength models presented in this paper. This is because, with the strong adhesives currently available and with appropriate surface preparation for the concrete substrate, debonding failures along the physical interfaces between the adhesive and the concrete and between the adhesive and the FRP plate are generally not critical.
Debonding may initiate at a flexural or flexural-shear crack in the high moment region and then propagates towards one of the plate ends (Figure 3a). This debonding failure mode is commonly referred to as intermediate crack (IC) induced interfacial debonding (or simply IC debonding) (Teng et al. 2002, 2003; Lu et al. 2007; Chen et al. 2006). Debonding may also occur at or near a plate end (i.e. plate end debonding failures) in four different modes: (a) critical diagonal crack (CDC) debonding (Figure 4b) (Oehlers and Seracino 2004), (b) CDC debonding with concrete cover separation (Figure 4c) (Yao and Teng 2007), (c) concrete cover separation (Figures 4d and 4e) (Teng et al. 2002), and (d) plate end interfacial debonding (Figure 4f) (Teng et al. 2002).

IC Debonding

When a major flexural or flexural-shear crack is formed in the concrete, the need to accommodate the large local strain concentration at the crack leads to immediate but very localized debonding of the FRP plate from the concrete in the close vicinity of the crack, but this localized debonding is not yet able to propagate. The tensile stresses released by the cracked concrete are transferred to the FRP plate and steel rebars, so high local interfacial stresses between the FRP plate and the concrete are induced near the crack. As the applied loading increases further, the tensile stresses in the plate and hence the interfacial stresses between the FRP plate and the concrete near the crack also increase. When these stresses reach critical values, debonding starts to propagate towards one of the plate ends, generally the nearer end where the stress gradient in the plate is higher.

Figure 5. FRP-plated RC beam: IC debonding
A typical picture of flexural crack-induced debonding is shown in Figure 5, which shows that a thin layer of concrete remained attached to the plate suggesting that failure occurred in the concrete adjacent to the adhesive-to-concrete interface. IC debonding failures are more likely to occur in shallow beams and are, in general, more ductile than plate end debonding failures.

**Concrete Cover Separation**

Concrete cover separation involves crack propagation along the level of the steel tension reinforcement. Failure of the concrete cover is initiated by the formation of a crack near the plate end. The crack propagates to and then along the level of the steel tension reinforcement, resulting in the separation of the concrete cover. As the failure occurs away from the bondline, this is not a debonding failure mode in strict terms, although it is closely associated with stress concentration near the ends of the bonded plate. A typical picture of a cover separation failure is shown in Figure 6. The cover separation failure mode is a rather brittle failure mode.

![Figure 6 FRP-plated RC beam: concrete cover separation](image)

**Plate-End Interfacial Debonding**

A debonding failure of this form is initiated by high interfacial shear and normal stresses near the end of the plate that exceed the strength of the weakest element, generally the concrete. Debonding initiates at the plate end and propagates towards the middle of the beam (Figures 4f and 7). This failure mode is only likely to occur when the plate is significantly narrower than the beam section, as otherwise, failure tends to be by concrete cover separation (i.e. the steel bars-concrete interface controls the failure instead).

![Figure 7. FRP-plated RC beam: plate end interfacial debonding](image)

**CDC Debonding**

This mode of debonding failure occurs in flexurally-strengthened beams where the plate end is located in a zone of high shear force but low moment (e.g. a plate end near the support of a simply-supported beam) and the amount of steel shear reinforcement is limited. In such beams, a major diagonal shear crack (critical diagonal crack, or CDC) forms and intersects the FRP plate, generally near the plate end. As the crack widens, high
interfacial stresses between the plate and the concrete are induced, leading to the eventual failure of the beam by debonding of the plate from the concrete; the debonding crack propagates from the crack towards the plate end (Figure 8).

In a beam with a larger amount of steel shear reinforcement, multiple shear cracks of smaller widths instead of a single major shear crack dominate the behaviour, so CDC debonding is much less likely. Instead, cover separation takes over as the controlling debonding failure mode. In other cases, particularly when the plate end is very close to the zero-moment location, CDC debonding leads only to the local detachment of the plate end, but the beam is able to resist higher loads until cover separation occurs (Figure 4c). The local detachment due to CDC debonding effectively moves the plate end to a new location with a larger moment, and cover separation then starts from this “new end”. The CDC failure mode is thus related to the cover separation failure mode. If a flexurally-strengthened beam is also shear-strengthened with U-jackets to ensure that the shear strength remains greater than the flexural strength, the CDC debonding failure mode may be effectively suppressed.

DEBONDING FAILURE MODES OF SHEAR-STRENGTHENED RC BEAMS

The shear failure process of FRP-strengthened RC beams involves the development of either a single major diagonal shear crack or a number of diagonal shear cracks, similar to normal RC beams without FRP strengthening (Chen and Teng 2003a, 2003b). For ease of description, the existence of a single major diagonal shear crack (the critical shear crack) is assumed whenever necessary. This treatment is conservative because recent research has shown that the existence of multiple cracks is beneficial for the development of the maximum debonding stress in FRP (Teng et al. 2006; Chen et al. 2007). Eventual failure of almost all test beams occurred in one of the two main failure modes: tensile rupture of the FRP and debonding of the FRP from the concrete. The FRP rupture failure mode has been observed in almost all tests on beams with complete FRP wraps and in some tests on beams with FRP U-jackets, while the debonding failure mode has been observed in almost all tests on beams with FRP side strips and most tests on beams with FRP U-jackets. Generally, both failure modes start with a debonding propagation process from the critical shear crack. Tensile rupture starts in the most highly-stressed FRP strip, followed rapidly by the rupture of other FRP strips intersected by the critical shear crack. In beams with complete FRP wraps, it is also common that many of the FRP strips intersected by the critical shear crack have debonded from the sides over the full height of the beam before tensile rupture failure occurs. In beams whose failure is by debonding of the FRP from the RC beam, failure involves a process of sequential debonding of FRP strips starting from the most vulnerable strip (Figure 9).
OTHER ASPECTS OF DEBONDING

The risk of debonding is increased by a number of factors associated with the quality of on-site application. These include poor workmanship and the use of inferior adhesives. The effects of these factors can be minimized if due care is exercised in the application process to ensure that debonding failure is controlled by concrete. In addition, small unevenness of the concrete surface may cause localized debonding of the FRP plate but it is unlikely to cause the FRP plate to separate completely from the concrete member. Mechanical anchors and FRP U-jackets can be used in soffit plated beams to prevent plate end debonding. The latter may be used for shear strengthening at the same time. For beams shear strengthened with FRP U-jackets and side strips, mechanical anchors can also be used to suppress FRP debonding failure, and thus change the failure mode from debonding to rupture. However, care needs to be exercised to avoid local failure adjacent to the anchors.

DEBONDING STRENGTH MODELS FOR FLEXURALLY-STRENGTHENED RC BEAMS

Plate End Debonding

Many factors control the likeliness of a particular plate end debonding failure mode for a given plated RC beam. For example, for an RC beam with a relatively low level of internal steel shear reinforcement, each of the plate end debonding modes (Figure 4) may become critical when the plate length or width is varied. When the distance between a plate end and the adjacent beam support (plate end distance) is very small, a CDC may form, causing a CDC debonding failure of the beam (Figure 4b). If the plate end distance is increased, the CDC may fall outside the plated region, and only concrete cover separation is observed (Figure 4d). Between these two modes, CDC debonding followed by concrete cover separation (Figure 4c) may occur; this mode is critical if the CDC debonding failure load is lower than the shear resistance of the original RC beam as well as the cover separation failure load so that the load can still be increased following CDC debonding. As the plate end moves further away from the support, the cover separation mode remains the controlling mode, and the plate end crack that appears prior to crack propagation along the level of steel tension reinforcement becomes increasingly vertical (Smith and Teng 2003). For the extreme case of a plate end in the pure bending region, the plate end crack is basically vertical (Figure 4e). For any given plate end position, if the plate width is sufficiently small compared with that of the RC beam, the interface between the soffit plate and the RC beam becomes a more critical plane than the interface between the steel tension bars and the concrete, and plate end interfacial debonding (Figure 4f) becomes the critical mode. However, this mode rarely occurs when the RC beam and the bonded plate have similar widths.

Given the larger variety of parameters that govern plate end debonding failures, the development of a reliable strength model is not a simple task. The recent model by Teng and Yao (2007) is the only model that appears to cover all the variations. The model caters for any combination of plate end moment and shear force via the following interaction curve:

\[
\left( \frac{V_{\text{db,end}}}{V_{\text{db,s}}} \right)^2 + \left( \frac{M_{\text{db,end}}}{M_{\text{db,f}}} \right)^2 = 1.0
\]

(1)

where \( V_{\text{db,end}} \) and \( M_{\text{db,end}} \) are the plate end shear force and the plate end moment at debonding respectively, \( M_{\text{db,f}} \) is the flexural debonding moment, and \( V_{\text{db,s}} \) is the shear debonding force.

The flexural debonding moment, which is the bending moment that causes debonding of a plate end located in the pure bending zone of a beam, is found from

\[
M_{\text{db,f}} = \frac{0.488M_{\nu,0}}{(\alpha_{\text{flex}}\alpha_{\text{axial}}\alpha_{\nu})_{\nu,0}} \leq M_{\nu,0}
\]

(2)

where \( \alpha_{\text{flex}} \), \( \alpha_{\text{axial}} \) and \( \alpha_{\nu} \) are three dimensionless parameters defined by

\[
\alpha_{\text{flex}} = \frac{(EI)_{c,frp} - (EI)_{c,0}}{(EI)_{c,0}}
\]

(3)

\[
\alpha_{\text{axial}} = \frac{E_{frp}t_{frp}}{E_{c}d_{c}}
\]

(4)

and

\[
\alpha_{\nu} = \frac{b_{c}}{b_{pp}}, \quad \frac{b_{c}}{b_{pp}} \leq 3
\]

(5)

where \( (EI)_{c,frp} \) and \( (EI)_{c,0} \) are the flexural rigidities of the cracked section with and without an FRP plate respectively; \( t_{frp} \) and \( E_{frp} \) are the width, thickness and elastic modulus of the FRP plate respectively; \( E_{c} \) is the elastic modulus of concrete, \( b_{c} \) and \( d \) are the width and effective depth of the RC beam respectively; and \( M_{\nu,0} \) is the theoretical ultimate moment of the unplated section which is also the upper bound of the flexural debonding moment \( M_{\text{db,f}} \).
The shear debonding force $V_{db,s}$, which is the shear force causing debonding of a plate end located in a region of (nearly) zero moment, can be found from

$$V_{db,s} = V_c + \varepsilon_{ve} \overline{V}$$

with $\varepsilon_{ve} \leq \varepsilon_y = \frac{f_t}{E_c}$.

where $V_c$ and $\varepsilon_{ve} \overline{V}$ are the contributions of the concrete and the internal steel shear reinforcement to the shear capacity of the beam respectively, and $\overline{V}$ is the shear force carried by the steel shear reinforcement per unit strain, i.e.

$$\overline{V} = A_s E_s d / s_t$$

where $A_s$, $E_s$, and $s_t$ are the total cross-sectional area of the two legs of each stirrup, the elastic modulus and the longitudinal spacing of the stirrups respectively. In Eq. 6, $\varepsilon_{ve}$ is the strain in the steel shear reinforcement, referred to here as the effective strain, and this effective strain may be well below the yield strain of the steel shear reinforcement $\varepsilon_y$. It should be noted that the bonded tension-face plate also makes a small contribution to the shear debonding force $V_{db,s}$ but this contribution is small and is ignored in this debonding strength model.

Based on available test results, Teng and Yao (2007) proposed that

$$\alpha_{flex} = \frac{2}{\alpha_w}$$

where $\alpha_{flex}$ and $\alpha_w$ are given by Eqs 3 and 5 respectively, while the other two dimensionless parameters are defined by

$$\alpha_E = \frac{E_{frp}}{E_c}$$

and

$$\alpha_t = \left( \beta_{frp} / \beta_c \right)^3$$

For the predictions of $V_c$ in design, the design formula in any national code may be used.

IC Debonding

Two simple and reliable IC debonding strength models have been developed by Teng et al. (2003) and Lu et al. (2007). The first model (Teng et al. 2003) is a simple modification of the bond strength model developed by Chen and Teng (2001). Lu et al.’s (2007) model is based the results of an extensive finite element study. Chen et al. (2006) explored an alternative approach that is based on rigorous analytical work on the behaviour of the FRP-to-concrete interfaces between two adjacent cracks (Teng et al. 2006; Chen et al. 2007). Chen et al.’s (2006) approach therefore has the most sound mechanics basis and is more versatile (e.g. it is applicable to all loading conditions). Primary research presented in Chen et al. (2006) has shown this model to be promising, and further work is in progress.

All the three models mentioned above predict a stress or strain value in the FRP plate at which IC debonding is expected to occur. According to Lu et al.’s (2007) model, this debonding stress is given by

$$\sigma_{dwc} = 0.114 (4.41 - \alpha) \tau_{max} \sqrt{\frac{E_{frp}}{f_{frp}}}$$

$$\tau_{max} = 1.5 \beta_{frp} f_t$$

$$\alpha = 3.41 L_{wcd} / L_d$$

$$\beta_{frp} = \frac{2.25 - b_{frp} / b_c}{1.25 + b_{frp} / b_c}$$

where $f_t$ is the tensile strength of concrete, $L_d$ (mm) is the distance from the loaded section to the end of the FRP plate while $L_{wcd}$ (mm) is given by

$$L_{wcd} = 0.228 \sqrt{E_{frp} f_{frp}}$$

DEBONDING STRENGTH MODELS FOR SHEAR-STRENGTHENED RC BEAMS

Several different approaches have been used to predict the shear strength of FRP-strengthened RC beams. These include the modified shear friction method, the compression field theory, various truss models and the design code approach (Teng et al. 2004). However, the vast majority of existing research has adopted the design code approach which is discussed below. The total shear resistance of FRP-strengthened RC beams in this approach is
commonly assumed to be equal to the sum of the three components from concrete, internal steel shear reinforcement and external FRP shear reinforcement respectively. Consequently, the shear strength of an FRP-strengthened beam $V_n$ is given in the following form:

$$V_n = V_c + V_s + V_{fp}$$  \hspace{1cm} (16)$$

where $V_c$ is the contribution of concrete, $V_s$ is the contribution of steel stirrups and bent-up bars and $V_{fp}$ is the contribution of FRP. $V_c$ and $V_s$ may be calculated according to provisions in existing design codes. The contribution of FRP is found by truss analogy, similar to the determination of the contribution of steel shear reinforcement. Two parameters are important in determining the FRP contribution: the shear crack angle which is generally assumed to be 45° for design use and the average stress (or effective stress) in the FRP strips intersected by the critical shear crack. Different models differ mainly in the definition of this effective stress. It may be noted that the design code approach neglects the interactions between the external FRP and internal steel stirrups and concrete. The validity of this assumption has been questioned by several researchers (e.g. Teng et al. 2002; Denton et al. 2004; Qu et al. 2005; Mohamed Ali et al. 2006), but the approach is the least involved for design, most mature and appears to be conservative for design in general. The most advanced model for FRP debonding failure following the design code approach is probably that developed by Chen and Teng (2003b) which employed an accurate bond strength model (Chen and Teng 2001), leading to accurate predictions.

According to Chen and Teng (2003b), the contribution of the FRP to the shear strength of the RC beam for a general strengthening scheme with FRP strips of the same width bonded on both sides of the beam (Figure 10) and with an assumed critical shear crack of $\theta=45^\circ$, is given by

$$V_{fp} = \frac{2f_{fp,e}w_{fp}h_{fp,e}(\sin \beta + \cos \beta)}{s_{fp}}$$  \hspace{1cm} (17)$$

where $f_{fp,e}$ is the average stress of the FRP intersected by the shear crack at the ultimate limit state, $w_{fp}$ is the thickness of the FRP, $w_{fp}$ is the width of each individual FRP strip (perpendicular to the fibre orientation), $s_{fp}$ is the horizontal spacing of FRP strips (i.e. the centre-to-centre distance of FRP strips along the longitudinal axis of the beam), $\beta$ is the angle of the inclination of fibres in the FRP to the longitudinal axis of the beam (measured clockwise for the left side of the beam as shown in Figure 2), and $h_{fp,e}$ is the effective height of the FRP bonded on the web:

$$h_{fp,e} = z_b - z_t$$  \hspace{1cm} (18)$$

where $z_t$ and $z_b$ are the coordinates of the top and the bottom ends of the effective FRP (Figure 10):

$$z_t = d_{fp,t}$$  \hspace{1cm} (19)$$

$$z_b = 0.9d - (h - d_{fp})$$  \hspace{1cm} (20)$$

in which $d_{fp,t}$ is the distance from the compression face to the top end of the FRP (thus $d_{fp,t} = 0$ for complete wrapping), $h$ is the height of the beam, and $d_{fp}$ is the distance from the compression face to the lower end of the FRP. When FRP is bonded to the full height of the beam sides, Eq. 18 reduces to $h_{fp,e} = 0.9d$ as $z_t=0$ and $z_b=0.9d$.

The FRP stress distribution at debonding failure is non-uniform chiefly because the bond lengths of the FRP strips vary with the vertical position of the critical shear crack at a given section. Chen and Teng (2003b) expressed the average (or effective) stress in the FRP along the critical crack $f_{fp,e}$ at the ultimate limit state as

$$f_{fp,e} = D_{fp}\sigma_{fp,max}$$  \hspace{1cm} (21)$$

in which $\sigma_{fp,max}$ is the maximum stress that can be reached in the FRP intersected by the critical shear crack and $D_{fp}$ is the stress distribution factor.
\[
\sigma_{\text{frp, max}} = \min \left\{ \begin{array}{l}
\frac{f_{\text{frp}}}{\alpha \beta \sqrt{E_{\text{frp}}/f_c}} \\
\frac{2}{\pi \lambda} \left(1 - \cos \frac{\pi \lambda}{2}\right) \quad \text{if } \lambda \leq 1 \\
1 - \frac{\pi - 2}{\pi \lambda} \quad \text{if } \lambda > 1
\end{array} \right. 
\]

(22)

\[
D_{\text{frp}} = \left\{ \begin{array}{l}
\frac{\pi \lambda}{2} \quad \text{if } \lambda \geq 1 \\
\sin \frac{\pi \lambda}{2} \quad \text{if } \lambda < 1
\end{array} \right. 
\]

(23)

where the coefficient \(\alpha\) has the best fit value of 0.427 and the 95 percentile characteristic value of 0.315 for design based on Chen and Teng’s (2001) bond strength model, \(\beta_L\) reflects the effect of bond length and \(\beta_w\) the effect of the FRP-to-concrete width ratio. The expressions for \(\beta_L\) and \(\beta_w\) are

\[
\beta_L = \left\{ \begin{array}{l}
1 \quad \text{if } \lambda \geq 1 \\
\sin \frac{\pi \lambda}{2} \quad \text{if } \lambda < 1
\end{array} \right. 
\]

(24)

\[
\beta_w = \sqrt{\frac{2 - \frac{w_{\text{frp}}}{s_{\text{frp}} \sin \beta}}{1 + \frac{w_{\text{frp}}}{s_{\text{frp}} \sin \beta}}} \geq \frac{\sqrt{2}}{2}
\]

(25)

Note that \(w_{\text{frp}}/(s_{\text{frp}} \sin \beta)\) is less than 1 for FRP strips with gaps. It becomes 1 when no gap exists between FRP strips and for continuous sheets or plates, yielding the lower limit value of \(\sqrt{2}/2\) for \(\beta_w\). The normalised maximum bond length \(\lambda\), the maximum bond length \(L_{\text{max}}\) and the effective bond length \(L_e\) of the FRP strips are given by

\[
\lambda = \frac{L_{\text{max}}}{L_e}
\]

(26)

\[
L_{\text{max}} = \left\{ \begin{array}{l}
\frac{h_{\text{frp}, r}}{\sin \beta} \quad \text{for U jackets} \\
\frac{h_{\text{frp}, r}}{2 \sin \beta} \quad \text{for side plates}
\end{array} \right. 
\]

(27)

\[
L_e = \frac{E_{\text{frp}} f_{\text{frp}}}{\sqrt{f_c}}
\]

(28)

The number 2 appears in the denominator for side plates in Eq. 27 because the FRP strip with the maximum bond length appears at the lower end of the critical shear crack for U-jacketing but at the middle for side plates. Equation 23 is applicable to both U-jackets and side strips. The actual calculated values are different for these two cases even if the configuration of the bonded FRP is the same on the beam sides because the maximum bond length \(L_{\text{max}}\) for U-jackets is twice that for side strips (see Eq. 27).

**CONCLUDING REMARKS**

This paper has been concerned with debonding failures of RC beams strengthened in either flexure or shear with externally bonded FRP reinforcement. A systematic classification of possible debonding failure modes has been presented. The mechanisms and processes of the different debonding failure modes have been examined in detail. Furthermore, advanced strength models for the key debonding failure modes have been summarised. The materials presented in this paper may be directly applied in the practical design of FRP strengthening systems for RC beams and serve as a useful basis for the future development of design provisions in design codes and guidelines.
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