PLATE END DEBONDING FAILURES OF FRP - OR STEEL-PLATED RC BEAMS: A NEW STRENGTH MODEL

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ABSTRACT

RC beams strengthened in flexure by bonding an FRP or steel plate to the tension face can fail by debonding at or near the plate end in a number of different modes. This paper presents a simple, rationally-based predictive model for such plate end debonding failures. In this model, pure flexural debonding for a plate end located in a pure bending region and pure shear debonding for a plate end located in a high-shear zero (or low)-moment region are first dealt with. The general case of a plate end under the combined action of shear and bending is treated as the interaction of these two extreme conditions. The proposed model is shown to be accurate through comparisons with available test results. The model relates the debonding failure load to the shear capacity of the beam and a number of well-defined parameters, and can be easily incorporated in any national or international design codes and guidelines.

KEYWORDS

FRP, RC beams, flexural strengthening, debonding, strength model.

INTRODUCTION

Bonding of an FRP plate to the tension face of a reinforced concrete (RC) beam can significantly increase its flexural strength (Teng et al. 2002; Oehlers and Seracino 2004). For simplicity of description, the commonly studied case of simply-supported beams is referred to hereafter so that the plate is bonded to the soffit of the beam. Since FRP-plated and steel-plated RC beams behave in a similar manner when they fail by debonding, the scope of this paper covers both types of beams.

![Debonding failure modes of a plated RC beam](Image)

Figure 1 Debonding failure modes of a plated RC beam
FRP-plated or steel-plated RC beams often fail by debonding in one of several possible modes (Figure 1). Debonding may initiate at a flexural or flexural-shear crack in the high moment region and propagates towards one of the plate ends (Figure 1a). This debonding failure mode has been referred to as intermediate crack (IC) induced interfacial debonding (or simply IC debonding) (Teng et al. 2002, 2003; Lu et al. 2005). Debonding may also occur at or near a plate end in four different modes: (a) critical diagonal crack (CDC) debonding (Figure 1b) (Oehlers and Seracino 2004), (b) CDC debonding with concrete cover separation (Figure 1c) (Yao 2004), (c) concrete cover separation (Figures 1d and 1e) (Teng et al. 2002), and (d) plate end interfacial debonding (Figure 1f) (Teng et al. 2002). CDC debonding is induced by the formation of a major shear crack intersecting the plate near a plate end and propagates from the point of intersection to the plate end along the plate-beam interface. Concrete cover separation involves the tearing-off of the concrete cover along the level of the steel tension reinforcement starting from a plate end. Plate end interfacial debonding also starts at a plate end, and then propagates along the plate-beam interface. In all these debonding failure modes, failure occurs in the concrete, either adjacent to the adhesive layer (interfacial debonding) or at the level of the steel tension reinforcement (cover separation), provided the plate is bonded to the beam in an appropriate manner using a strong adhesive.

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 four plate end debonding modes 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 1b). If the plate end distance is increased, the CDC may fall outside the plated region, and only concrete cover separation is observed (Figure 1d). Between these two modes, the combined mode of CDC debonding and concrete cover separation (Figure 1c) 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 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 1e). For an arbitrary 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 1f) becomes the critical mode. However, this mode rarely occurs when the RC beam and the bonded plate have similar widths.

Many theoretical models for predicting plate end debonding failures have been proposed, although these models have often been proposed for only one or some of the plate end debonding failure modes shown in Figure 1. Smith and Teng (2002a, 2002b) focused on concrete cover separation and plate end interfacial debonding (Figures 1d and 1f), and reviewed and assessed all predictive models published prior to their work. They showed the deficiencies of those models and proposed a simple and conservative model based on the shear capacity approach which was found by them to be the most rational approach. In this approach, the plate end debonding load is related to the concrete component of the shear capacity of the RC beam, with or without an additional contribution from the internal steel shear reinforcement. Smith and Teng (2003) subsequently considered the interaction between the bending moment and the shear force acting at the plate end. For Smith and Teng’s (2003) approach to be used in design, it requires the predictions of debonding failure loads for two extreme cases of plate end location: (a) pure shear debonding when the plate end is at a...
location of little moment (e.g. near the support); and (b) pure flexural debonding when the plate end is located in a pure bending zone. Furthermore, the form of the shear-bending interaction equation also needs to be further examined. This paper extends the work of Smith and Teng (2003) by presenting accurate predictive equations for both shear debonding and flexural debonding, together with a more accurate shear-bending interaction curve.

**FLEXURAL DEBONDING**

**Test Database for Flexural Debonding**

Existing test data for the flexural debonding of RC beams bonded with a soffit plate within the pure bending region (Figure 1e) are very limited. Only eight tests, with sufficient details of the geometric and material parameters to enable their confident use, have been found from the existing literature (Oehlers 1992, Mohamed Ali et al. 2001, Smith and Teng 2003). With the ten tests from Yao (2004), a total of eighteen flexural debonding tests are thus available. These tests include 4 steel-plated beams and 14 FRP-plated beams and cover the significant parameters over a wide range (Teng and Yao 2005). In all these tests, the soffit plate was neither prestressed nor anchored in any form at its ends and the beam never experienced loading before being loaded to debonding failure.

**Oehlers and Moran’s Strength Model for Flexural Debonding**

Based on their own test results of steel-plated beams, Oehlers and Moran (1990) proposed the following predictive model for the flexural debonding moment \( M_{db,f} \) at the plate end:

\[
M_{db,f} = \frac{(EI)_{cr,p} f_{ct}}{0.901 E_p t_p}
\]

where \((EI)_{cr,p}\) is the flexural rigidity of the cracked plated section, \(f_{ct}\) is the concrete splitting tensile strength, \(E_p\) and \(t_p\) are respectively the elastic modulus and the thickness of the soffit plate. This model is the only flexural debonding strength model available in the published literature. While it offers reasonably close predictions of the test results from their research group, its predictions of the other test results of the database are inaccurate (Figure 2a). The model overestimates the flexural debonding moments of twelve out of the fourteen FRP-plated beams as well as the steel-plated beam tested by Yao (2004) (specimen SP-A) which had a low tension reinforcement ratio of 0.49%. Furthermore, for those RC beams bonded with a thin soffit plate \((t_p \leq 2\text{mm})\), the debonding moment predicted by Eq. 1 is far higher than the theoretical ultimate moment of the unplated section \(M_{u,0}\) (Teng and Yao 2005). This is unreasonable because for a plated beam with the critical plate end in the constant moment region, the latter is the upper bound of the former. It is therefore clear that Oehlers and Moran’s model (Eq. 1) cannot be used at least for FRP-plated RC beams in practice. Indeed, this model is inaccurate even for steel-plated beams different from those tested by the proposers’ group.

![Figure 2](image_url)

Figure 2 Flexural debonding moments: test results versus predictions of theoretical models
Proposed Strength Model for Flexural Debonding

A bonded plate is forced to deform with the original RC beam. This composite action leads to interfacial stresses at the plate-beam interface and at the steel bars-concrete cover interface. Provided the FRP plate is sufficiently wide relative to the beam, flexural debonding occurs along the latter interface when it can no longer resist the stresses induced by this composite action. The axial and bending rigidities of both the cracked RC section and the bonded plate are important parameters that determine the magnitudes of stresses at this interface. The cover thickness of concrete is also important as it affects the stresses at the steel bars-concrete interface for a given tensile force in the soffit plate. Furthermore, the width ratio between the plate and the RC beam is important as it affects directly the interfacial stresses between the beam and the concrete, and these interfacial stresses have a direct bearing on the initiation of cracking at the plate end. Based on these considerations, a detailed analysis of the results of the eighteen tests was conducted to identify the effect of each of the parameters that is likely to have a significant bearing on the flexural debonding strength. This analysis led to the following predictive model for flexural debonding of a plate end located in a pure bending region:

\[ M_{db,f} = \frac{0.488M_{u,0}}{(\alpha_{flex}\alpha_{axial}\alpha_{w})^{1/2}} \leq M_{u,0} \]  \hspace{1cm} (2)

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

\[ \alpha_{flex} = \frac{(EI)_{c,p} - (EI)_{c,0}}{(EI)_{c,0}} \]  \hspace{1cm} (3)

\[ \alpha_{axial} = \frac{E_{pt}b}{E_{d}d_{e}} \]  \hspace{1cm} (4)

and

\[ \alpha_{w} = \frac{b_{p}}{b_{e}}, \quad b_{p}/b_{e} \leq 3 \]  \hspace{1cm} (5)

where \((EI)_{c,p}\) and \((EI)_{c,0}\) are the flexural rigidities of the cracked section with and without a soffit plate respectively, \(E_{pt}\) is the axial rigidity per unit width of the soffit plate, \(d_{e}\) is the effective depth of the beam, and \(M_{u,0}\) is the theoretical ultimate moment of the unplated section which is also the upper bound of the flexural debonding moment \(M_{db,f}\). It is obvious that \(\alpha_{flex}\) reflects the effect of the contribution of the soffit plate to the flexural rigidity of the cracked section, \(\alpha_{axial}\) reflects the effect of the axial rigidity ratio, and \(\alpha_{w}\) reflects the effect of the width ratio. The limitation imposed on the width ratio reflects the limitation of the test data. These parameters indirectly reflect the effects of several other parameters. For example, the effects of concrete strength and steel tension reinforcement ratio are included in the theoretical ultimate moment of the unplated section \(M_{u,0}\).

Eq. 2 is a best-fit expression of the eighteen test results, with the limitation that \(M_{db,f}\) does not exceed \(M_{u,0}\). The test-to-predicted debonding moment ratios have an average value of 1.004 and a corresponding standard deviation of 0.094. Eq. 2 is therefore an accurate representation of these test results as seen in Figure 2b, and is applicable to both steel-plated and FRP-plated RC beams.

SHEAR DEBONDING

General

If the end of a soffit plate is subject to a shear force with little or no bending moment, then debonding is due to the shear force alone (pure shear debonding). A shear debonding strength model, which is an extension of the model proposed by Oehlers et al. (2004), is proposed here on the basis of the tests conducted at The Hong Kong Polytechnic University (Smith and Teng 2003; Yao 2004). Zhang (1997) developed an iterative process for the determination of the shear capacity of RC beams without internal shear reinforcement. Oehlers et al.’s (2004) model was an extension of Zhang’s (1997) model by including the contribution of the soffit plate. The shear debonding strength model proposed here extends Oehlers et al.’s (2004) model to include the contribution of the internal steel shear reinforcement.

Proposed Strength Model for Shear Debonding

The shear debonding strength can be represented by the debonding shear force \(V_{db,s}\) at a plate end located in a region of (nearly) zero moment. Test observations (Yao 2004) showed that shear debonding can be in one of the three modes shown in Figures 1b-1d and the debonding shear force lies between the shear resistance contributed by the concrete of the unplated beam and the full shear capacity of the plated beam. Therefore, the following equation is proposed to predict the shear debonding strength:

\[ V_{db,s} = V_{c} + V_{p} + \varepsilon_{w}V_{e} \]  \hspace{1cm} (6)
where $V_c$, $V_p$ and $e_{ve}$, $V_s$, $\epsilon$ are the contributions of the concrete, the soffit plate, and the internal shear reinforcement to the shear capacity of the beam respectively, and $V_e$ is the shear force carried by the steel shear reinforcement per unit strain, that is

$$ V_e = A_{sv} E_{sv} d_s / \epsilon $$

where $A_{sv}$, $E_{sv}$ and $\epsilon_s$ are the total cross-sectional area of the two legs of each stirrup, the elastic modulus and the longitudinal spacing of the stirrups. In Eq. 6, $e_{ve}$ 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 at debonding failure. For the predictions of $V_c + V_p$, it is proposed that Oehlers et al.’s prestress model (2004) be adopted.

Eight tests on plated RC beams in which the critical plate end distance was not greater than 50 mm were analysed to establish an expression for the effective strain $e_{ve}$. In deducing the experimental values of $e_{ve}$, the value of $V_c + V_p$ was found from a corresponding plated beam (specimen CS-CN) which had no internal shear reinforcement and failed by CDC debonding (Yao 2004), with due adjustment to account for the differences in the geometry and the concrete strength using Oehlers et al.’s prestress model (2004).

Based on this analysis, the best-fit expression for $e_{ve}$ for the eight tests is given by

$$ e_{ve} = \frac{10}{(\alpha_{flex} \alpha_{E} \alpha_t \alpha_w)^{1/2}} $$

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

$$ \alpha_{E} = E_p / E_c $$

and

$$ \alpha_t = \left(\frac{d_s}{d_c}\right)^{3} $$

It may be noted that $\alpha_{t gal}$ is now separated into $\alpha_{E}$ and $\alpha_t$ to better reflect the trends of the test results. Otherwise, the effective strain is assumed to depend similarly on the parameters which are important for flexural debonding. This assumption is based on the observation that shear debonding generally occurs after the formation of a significant shear crack which greatly increases the curvature near the plate end (Yao 2004). That is, the development of a cover separation failure, following the formation of a significant shear crack, also depends on the stresses required at the steel bars-concrete interface to maintain curvature compatibility between the RC beam and the bonded plate. A comparison of the effective strain values predicted using Eq. 8 with those deduced from the test results is shown in Figure 3. The ratios between the test and the predicted values of effective strain have an overall average value of 1.008 and a corresponding standard deviation of 0.224. Therefore, Eq. 8 is in good agreement with these results.

**Proposed Model versus Other Test Results**

A database of 38 tests was assembled from 13 existing studies on simply supported RC beams bonded with a soffit plate that terminated at a zero or nearly zero moment location (i.e. near the support of a simply-supported beam). This database together with the 8 tests used to establish Eq. 8 covers a large number of parameters over a wide range (Yao 2004).
In Figure 4, the results of the 46 tests available are compared with the predictions of Eq. 6 in conjunction with Eqs 7 and 8, where the shear capacity of the beam contributed by the concrete and the soffit plate $V_c + V_p$ for each of the 38 tests collected from the 13 existing studies was determined using Oehler et al.’s model (2004).

It is evident that the test results are in close agreement with the predictions, although Eqs. 6 to 8 are based only on the results of eight tests. Indeed, the proposed model leads to results which agree with the test results better than any of the existing models (Teng and Yao 2005). The ratios between the test and the predicted shear debonding strengths have an overall average value of 1.070 and a corresponding standard deviation of 0.200. More importantly, the proposed model is based on the correct failure mechanism and explicitly reflects the important contribution of the internal shear reinforcement to the shear debonding failure load.

It should be noted that Oehler’s et al.’s model (2004) for shear debonding differs from Zhang’s approach for the shear capacity of RC beams without internal steel shear reinforcement only in that the former includes the contribution of the axial force in the soffit plate to the shear capacity. In Figure 4, results obtained from Eq. 6 with $V_p$ ignored are also shown. It is clear that the contribution of $V_p$ is generally small, with differences ranging from 1.6% to 11.9% and being 5% on average. By contrast, if contribution of the soffit plate is included but the contribution of the internal shear reinforcement is ignored, which reduces the present model to Oehler et al.’s (2004) model, the resulting differences are much larger (Figure 5), ranging from 2.4% to 46.5% and being 13.2% on average. These differences are small for small beams, but become larger when the beam dimensions become more realistic.

<table>
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<th>$V_{db,s}$ (kN)</th>
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Figure 5 Contribution of steel shear reinforcement to debonding strength

Figure 6 Interaction between bending moment and shear force at the plate end at debonding

Based on the comparisons shown in Figures 4 and 5, for practical applications of the proposed model for shear debonding, the expression of $V_c$ (Eq. 6) can be replaced by the corresponding expression in any national code for concrete structures, while $V_p$ can be ignored. These simplifications eliminate the iterative process of Zhang’s (1997) and Oehler’s et al.’s (2004) models and yields more conservative predictions (Teng and Yao 2005).

**INTERACTION BETWEEN BENDING AND SHEAR**

Oehler’s (1992) and Smith and Teng (2003) have both shown that significant interaction between the bending moment and the shear force at a plate end exists in a plate end debonding failure. To further examine this interaction, the existing literature was carefully examined to find test data which can be used to study this interaction. Only limited test results from 6 separate research groups were found from the existing literature (Teng and Yao 2005). Based on these test results, it was found that a circular interaction curve provides a better representation of the experimental results than the bi-linear curve suggested by Smith and Teng (2003). This circular interaction curve is defined by the following equation and compared with the test results in Figure 6:

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

where $V_{db,end}$ and $M_{db,end}$ are the plate end shear force and the plate end moment at debonding, respectively. In making the predictions using Eq. 11 for Figure 6, the flexural debonding moment $M_{db,f}$ for each set of test beams was found from the proposed model (Eq. 2), while the shear debonding force $V_{db,s}$ was deduced using Eq. 11 from the test results of the beam with the smallest plate end distance in the same set.

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CONCLUSIONS

Based on insights gained from recent tests at The Hong Kong Polytechnic University and existing work by other researchers, a simple and rationally-based predictive model for plate end debonding failures in FRP-plated and steel-plated RC beams has been proposed. In this model, pure flexural debonding for a plate end located in a pure bending region and pure shear debonding for a plate end located in a high-shear zero (or low)-moment region are first dealt with. The general case of a plate end under the combined action of shear and bending is treated as the interaction of these two extreme conditions. The proposed model reflects explicitly the contributions of both the internal steel shear reinforcement and the bonded plate to the debonding failure load and has been shown to be accurate through comparisons with available test results. Within the general framework of the proposed model, future refinements of the model to improve its accuracy of predictions can be easily achieved when more results from tests and/or numerical predictions become available. The model relates the debonding failure load to the shear capacity of the beam and a number of well-defined parameters, and can be easily incorporated in any national or international design codes and guidelines.

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