ANALYTICAL MODELING OF FLEXURAL DEBONDING IN CFRP STRENGTHENED REINFORCED OR PRESTRESSED CONCRETE BEAMS

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1 INTRODUCTION

Due to a deteriorating infrastructure many buildings and bridges are in need of rehabilitation. Strengthening of existing structures using lightweight composite materials is becoming widespread due to their ease of installation and competitive pricing compared to traditional methods. Fiber Reinforced Polymer (FRP) strengthened reinforced or prestressed concrete beams often fail in flexure, due to concrete crushing or FRP rupture. This type of failure can be well predicted using a cracked section analysis of the strengthened section using the specified FRP material properties from the manufacturer [1]. The behavior under fatigue loading of FRP strengthened beams is often controlled by the stress range of the internal steel reinforcement, which should be kept within prescribed limits [2].

Interface crack propagation of FRP strengthened reinforced or prestressed concrete flexural members often occurs from bridged intermediate cracks towards the supports. This type of FRP delamination is commonly termed intermediate crack (IC) debonding and is common in flexural members with high shear span-to-depth ratio. In the literature many analytical models have been proposed to predict the strain in FRP material at IC debonding (e.g. [3-7]), and several have been codified (e.g. [8-10]). An extensive review of these models can be found elsewhere [11]. In this paper an experimental program is described where six 9.18 m prestressed concrete bridge girders were tested to evaluate the bond characteristics CFRP strengthening systems. Four of the beams failed due to IC debonding, one failed due to FRP rupture, and the control failed due to concrete crushing. An analytical model is proposed which characterizes the interface shear stress along the beam as coming from two sources: the applied loading and stress concentrations at the flexural cracks. The scope of the proposed model is discussed here, and its effectiveness in predicting the failure modes of beams that are partially prestressed, FRP wrapped around the soffit, or in the presence of debonding mitigation such as transverse U-wraps.

2 EXPERIMENTAL STUDY

A total of six girders were tested as part of the experimental study. Three girders were strengthened with precured CFRP strips (EB1S, EB1SB, and EB1SB2), and two girders were strengthened with CFRP wet lay-up sheets (EB8SB and EB9SB). An unstrengthened control girder (CS), and a girder strengthened with precured CFRP strips and debonding mitigation (EB1S), were tested as part of an earlier experimental program and further details can be found elsewhere [11].

2.1 Test Girders

Several precast prestressed C-Channel type bridge girders were tested as part of this research. The 9.18 m long girders were prestressed with 1724 MPa stress-relieved 7-wire prestressing strands. Five strands were in each web, of which the top three were harped with a hold down point located at midspan as shown in Figure 1. Two types of Carbon FRP (CFRP) material was used in this research: precured strips and wet lay-up sheets. Girders EB1S, EB1SB, and EB1SB2 were all strengthened with one 51 mm precured strip per web. Girder EB8SB was strengthened with two 51 mm sheets per web and girder EB9SB was strengthened with four 51 mm sheets per web. All of the strengthened girders had debonding mitigation, consisting of ten transverse CFRP wet lay-up U-wraps (5 on each web) on one side only to promote debonding on the instrumented side, except girder EB1S which had
the same debonding mitigation placed throughout the length of the girder. Details of the CFRP strengthening is shown in Figure 1 and summarized in Table 1. All constitutive materials were tested to determine their stress-strain behavior in accordance with the appropriate ASTM standard and the test results are shown in Table 1. All girders were tested using a 490 kN MTS hydraulic actuator mounted to a steel frame placed at the midspan of the girder. The load was applied using a 254 x 508 mm steel plate. In order to simulate field conditions, the girder was supported at both ends on neoprene pad. The width of the neoprene pad was 216 mm which provided a clear span of 8117 mm for each tested girder. The girders were tested first up to the cracking load, unloaded and reloaded to failure. This allowed observation of the crack re-opening load which aided in the computation of the effective prestress force. The displacement behavior of the girders during testing was monitored using two string potentiometers placed at midspan, and two linear potentiometers were used to measure vertical displacement over the supports. The compressive strain in the concrete was measured using a combination of PI gauges (a strain gauge mounted to a spring plate) and two electric resistance strain gauges located at midspan. PI gauges were placed at various locations at the level of the lowest prestressing strand to measure the crack width and to determine the strain profile along the depth of the girder. The tensile strain profile in the CFRP reinforcement was measured using numerous electric resistance strain gauges placed at various locations along the length of the girder.

Fig. 1 Girder reinforcing and CFRP strengthening details.
Table 1  Summary of material properties and experimental results.

<table>
<thead>
<tr>
<th>Specimen Designation</th>
<th>EB1SB</th>
<th>EB1SB2</th>
<th>EB8SB</th>
<th>EB9SB</th>
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<tr>
<td>Concrete Strength, (MPa)*</td>
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<td>35.7</td>
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<td>Modulus of Elasticity of FRP, GPa*</td>
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<tr>
<td>Ultimate strength of prestressing, MPa*</td>
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<td>1812</td>
<td>1813</td>
<td>1813</td>
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<tr>
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<td>153.5</td>
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<td>1163</td>
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<td>0.79</td>
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<td>IC</td>
<td>IC</td>
<td>Rupture</td>
<td>IC then Rupture</td>
</tr>
</tbody>
</table>

* From material testing  
** IC = Intermediate Crack Debonding

2.2 Discussion of test results

A discussion of the experimental results of the bond study is discussed in this section. Detailed discussion of all the tested girders is discussed elsewhere [1]. The structural behavior of the girders up to debonding failure was similar to other girders tested [11]. A summary of the test results of the bond girders is shown in Table 1. The IC debonding failure of girder EB1SB is shown in Figure 2.

Girder EB1S had debonding mitigation placed along the length of the girder, which reduced the propensity for debonding failure and increased the ultimate load and displacement at ultimate. For the other strengthened girders, the loads and ultimate displacements were lower due to premature debonding failures, which was intentionally implemented to study the failure mechanism. The applied load versus midspan displacement is shown in Figure 3 for all the strengthened girders, along with the unstrengthened control girder (CS). All of the girders strengthened with CFRP failed due to IC debonding except girder EB8SB which failed due to rupture of FRP. Another girder, EB9SB failed due to IC debonding followed by rupture of FRP. The only girder with debonding mitigation (EB1S) showed considerably more deflection than the other strengthened girders. The girder with the largest axial stiffness of FRP (EB9SB) achieved the highest measured load value, an increase of 20 percent compared to the control girder.
The girders tested as part of the bond study were instrumented with numerous electrical resistance strain gauges affixed to the CFRP material. From continuous monitoring of the output of the strain gauges a strain profile was established along the length of the girder for various load levels. The strain profile for girder EB1SB, which failed due to intermediate crack debonding, is shown in Figure 4, where the dark lines represent the load of flexural cracking and debonding failure. The figure indicates that the tensile strain in the CFRP material at service load levels are far below strains to cause debonding failure. Debonding mitigation was provided for girder EB1SB on one side (the right side of the figure), but the effect of the transverse U-wraps on the tensile strain profile of the
longitudinal CFRP is unclear. Girders EB1SB2, and EB8SB had similar axial stiffnesses of longitudinal CFRP resulting in similar strain distributions. From the tensile strain profiles it can be shown that externally bonded wet lay-up type systems have a more uneven distribution of strain due to the conformation of the strengthening material to the soffit of the beam.

![Fig. 4 Tensile strain in CFRP versus length along girder for girder EB1SB.](image)

Using the measured tensile strain in the CFRP along the span, the interface shear stress, \(\tau(x)\), along the same length can be evaluated using the following equation:

\[
\tau(x) = \frac{d}{dx} \left[ K_p (x) \varepsilon_p (x) \right]
\]

(1)

where \( \frac{d}{dx}[\varepsilon_p(x)] \) is the change in plate strain along the length of the beam \( x \) and \( K_p \) is the axial stiffness of the plating material per unit width. From the strain profile plots of each of the girders, the slope of the tensile strain can be easily calculated between the values of measured strain. This quantity multiplied by the axial stiffness of the material will give the interface shear stress along the length of the beam. The interface shear stress versus the length along the girder for girder EB1SB is shown in Figure 5. The uniform distribution of strain resulted in an interface shear stress distribution which was positive for over half the girder with a maximum value of 1.0 MPa. The variability in these plots is due to the complex cracking behavior of reinforced/prestressed concrete and the resulting stress concentrations which occur around the toes of the flexural cracks. From this plot a general trend can be shown where the maximum interface shear stress at debonding failure will likely occur at the point along the girder at which first yielding of the prestressing strands is occurring, approximately 3000 to 4000 mm from the support.
3 ANALYTICAL MODEL

3.1 Analytical Model

The proposed analytical model is based on mechanics, and calibrated against an extensive experimental database of IC debonding failures. Various failure criteria were examined in the construction of the model, and an equation from Matthys [12] was adopted. A check for rupture of FRP is also provided in the analytical model, using an equation from Teng [6]. The initial strain in the beam soffit has been incorporated into the CFRP strain during calculation of the FRP system contribution to sectional strength. This strain is normally additive for prestressed concrete members and subtractive for reinforced concrete members.

A plated section with perfect bond along the plate-to-concrete interface will have an interface shear stress ($\tau_w$) equal to the change in plate strain along the length of the beam multiplied by the axial stiffness of the plating material per unit width. The maximum interface shear stress due to applied loading will occur at a location where the internal tensile steel has yielded. Since the peeling stresses will not significantly affect the behavior away from the supports, there exists only forward shear mode (Mode II) crack deformation on the FRP-to-concrete interface due to the applied loading. With the assumption of perfect bond, failure due to debonding will occur when the interface shear stress reaches the shear strength of the interface. Usually the shear strength of the structural adhesive is greater than that of the concrete, so failure will occur along the concrete interface.

FRP plated regular reinforced and prestressed concrete beams will exhibit flexural behavior when the shear span-to-depth (a/d) ratio is approximately 2.5 or greater, except in rare cases such as when there is exceptionally high prestress force. It is assumed that the beam being strengthened in flexure has sufficient shear capacity to handle the higher loads and large shear crack deformations will not control the failure. The interface shear stress at the toes of the flexural cracks due to stress concentration is related to the fracture energy of the weakest material at the interface. With quality workmanship and critical selection of an acceptable externally bonded FRP system, failure will propagate initially along the concrete interface. The profile of the shear stress due to stress concentration around the toe of a flexural crack is similar to a single lap-shear test, and can be approximated using equations found elsewhere [13]. However, it is the magnitude of the interface shear stress which is important, and this is not predicted using these equations. The shear stress at the FRP-to-concrete interface ($\tau_i$) is induced from two sources: the applied loading ($\tau_w$) and stress...
concentrations at the toes of flexural cracks ($\tau_{sc}$). The value will be at a maximum in high moment regions where the crack opening displacement has the greatest magnitude. The applied moment, tensile force in FRP, and interface shear stress distribution is shown in Figure 6 for the loading configuration shown.

![Diagram of tensile force in FRP and interface shear stress](image)

**Fig. 6** Interfacial stresses in plated RC beam

The actual distribution of interface shear stress along the length of a plated beam is complex. The interface shear stress due to applied loading varies depending on the change in CFRP tensile force along the length of the beam, and stress concentrations at the toes of the flexural cracks result in complex behavior. There is a need for a simplified approach for determining the interface shear stress for design. It is proposed that the maximum interface shear stress due to applied loading ($\tau_{wmax}$) be determined from the following equation, related to the increase of applied moment from $M_y$ to $M_{db}$.

The equation predicts the maximum interface shear stress as in the region along the beam from first yielding of the prestressing strands to location of maximum moment:

$$\tau_{wmax} = nE_{ft} \frac{\epsilon_{db} - \epsilon_{fy}}{a - x_y}$$

where $nE_{ft}$ is the axial stiffness of FRP material per unit width, $\epsilon_{db}$ is the strain in the FRP at intermediate debonding failure at a moment of $M_{db}$, $\epsilon_{fy}$ is the tensile strain in the FRP at first yielding of internal tensile steel at a moment of $M_y$, $x_y$ is the distance from the support to the location of first yielding of internal tensile steel. For three and four point bending, $x_y$ is equal to:

$$x_y = \frac{a M_y}{M_{db}}$$

where $a$ is the shear span of the beam and is equal to the distance from the support to the section of maximum moment. For unsymmetrical loading, the shorter distance should be used. The distance $x_y$ can also be found easily for beams with other loading scenarios by considering the shape of the moment diagram. Equations similar to Equation 2 can be used to calculate the interface shear stress due to applied loading for two other regions: 1) the region between the supports and the location of flexural cracking, and 2) the region between the instance of first flexural cracking and yielding of the internal tensile steel. This is illustrated in Figure 7, where the proposed interface shear stress blocks are plotted versus the length along the member.

The analytical model was calibrated from a database of experimental intermediate crack (IC) debonding failures. The parameter that was calibrated was the interface shear stress due to stress concentration ($\tau_{scmax}$). The experimental database is described elsewhere [11].

The equation for $\tau_{scmax}$ which gives the best correlation between the analytical model and the experimental database is:

$$\tau_{scmax} = 2.15 \left( 1.1 - \frac{M_y}{M_{db}} \right) \sqrt{j_c}$$

For design, Equation 4 was modified to give a probability of exceedance of 5 percent as follows:
Max 31.1
\[ \varepsilon_{\text{sc}} = 3 \left( 1.1 - \frac{M_y}{M_{\text{db}}} \right) \sqrt{f_c} \] (5)

**Fig. 7** Interfacial stresses in plated RC beam

In several beams of the experimental database, the failure mode was not intermediate crack (IC) debonding, but rupture of FRP. The measured value of maximum strain near midspan prior to rupture was lower than that determined from FRP tensile tests, due to the presence of stress concentrations at the toes of the flexural cracks. Therefore it is believed that the strain due to these stress concentrations\( (\varepsilon_{\text{sc}}) \) caused rupture at a value of tensile strain assuming perfect bond\( (\varepsilon_{\text{db}}) \) lower than the ultimate rupture strain found during material testing\( (\varepsilon_u) \). As a result, a check against rupture of FRP due to stress concentrations at flexural cracks must be performed. For its simplicity, the equation of Teng [6] is adopted in the model with a surface distribution factor of 0.5:

\[ \varepsilon_{\text{db}} + 0.114 \frac{\tau_{\text{sc,max}}}{\sqrt{E_f t_f}} \leq \varepsilon_u \] (6)

where \( \varepsilon_{\text{db}} \) is the design rupture strain of the FRP material, after application of appropriate environmental reduction factors.

**Summary of Design Model**

1. Calculate the moment resistance corresponding to yielding of the tensile steel reinforcement, \( M_y \), and the corresponding strain level in the FRP material\( (\varepsilon_{\text{fib}}) \).
2. Assume a value for the strain in CFRP at failure due to intermediate crack (IC) debonding\( (\varepsilon_{\text{db}}) \).
3. Calculate the nominal moment resistance of the section\( (M_{\text{db}}) \) at debonding failure with the strain assumed in Step 2.
4. Determine the interface shear stress due to the increase of the load from the yielding stage to the assumed failure stage, \( \tau_{\text{wmax}} \) from Equation 2.
5. Determine the maximum interface shear stress due to stress concentration\( (\tau_{\text{sc,max}}) \) using Equation 5.
6. Calculate the maximum interface shear stress\( \tau_i = \tau_{\text{wmax}} + \tau_{\text{sc,max}} \).
7. Revise the value of IC debonding strain until \( \tau_{\text{wmax}} \) is equal to the failure criterion\( (\tau_{\text{cmax}}) \) which is related to the tensile strength of concrete as \( \tau_{\text{cmax}} = 1.8 \times (0.63 f_c)^{0.5} \).
8. Using Equation 6, perform check against rupture of FRP.

**3.2 Scope of Analytical Model**

Through an examination of the range of parameters in the IC debonding database and based on experimental results from other types of FRP configurations, the scope of the proposed analytical model can be ascertained. The applicability of the model to several different scenarios is discussed in this section including FRP wrapping around the beam soffit, side-plating, multiple layers of reinforcement, FRP extension to supports, and the configuration of internal tensile steel reinforcements.
In some instances externally bonding CFRP plates to the sides of a beam might be useful, especially in building structures where limited access is available to the bottom of the beam due to placement of ducting and electrical conduits. In Breña [14] several specimens were tested with CFRP plates externally bonded to the sides of the beam. The authors observed mixed results; in some beams this configuration resulted in increased debonding strain values and in others it reduced the maximum induced strain in the CFRP material. It is recommended that further research is needed before the proposed analytical model can be used for this case.

Wrapping the tension-side soffit of the beam with FRP can be an effective way to mitigate debonding. In one research project several beams were strengthened with FRP wrapped around their soffits and transverse CFRP U-wraps provided throughout the length [11]. IC debonding did not cause failure in any of these girders but they were analyzed using the proposed mean model. The model was conservative in every instance, and in several cases predicted the correct failure mode. It is possible that an FRP strengthening scheme with wrapping could induce IC debonding, especially if transverse U-wraps are not provided.

A girder strengthened with near surface mounted strips [11] was analyzed using the proposed model. The FRP configuration was input into the model using several different approaches and does a poor job of predicting the debonding strain when the strip is oriented in the vertical "as-built" configuration, but gives a reasonably conservative prediction if it is rotated 90 degrees. It is recommended that if NSM FRP systems are used, other predictive models should be used [15,16].

The analytical model provides good results in predicting IC debonding resistances when the FRP is extended to near the supports. Within the IC debonding database, the lowest ratio of the FRP span within the shear span to the shear span itself is 0.66. Unless there are obstructions which prevent the FRP to being extending to near the supports, this should be done in every case.

With members that are partially prestressed (with both regular mild steel reinforcement and prestressing steel), the interface shear stress should be calculated in a more complex manner than the method presented here. The interface shear stress due to the applied loading will depend on which material dominates the behavior of the section. When significant non-linear behavior of the internal steel is present, force redistribution will increase demand on the CFRP strengthening increasing the slope of the CFRP tensile strain envelope and the interface shear stress.

Another assumption that has been made in the proposed analytical model is the lumping together of the internal tensile steel for ease of calculations. In a push-over type of analysis, the applied moment to yield the internal steel reinforcing will be less if the layers are not grouped together, i.e. the lowest layer will yield before the others. In the C-Channel type prestressed concrete girder examined as part of this research, there are five layers of prestressing reinforcement that are distributed in the web of the girder. However, the yield moment calculated using a detailed cracked section analysis considering the various layers of steel and the yield moment calculated using a simple stress block approach with the prestressing strands lumped together is nearly identical. Therefore the lumping together of the internal steel should not greatly influence the validity of the model.

4 CONCLUSIONS

An analytical tool was developed which accurately predicts the intermediate crack (IC) debonding strain for reinforced and prestressed concrete members flexurally strengthened with FRP materials.

1. From experimental observations and analytical modeling, it was found that the interface shear stress along the length of the strengthened member comes from two distinct sources: the applied loading and stress concentrations at the toes of flexural cracks.
2. From various analyses of the strengthened reinforced concrete member, it was found that the interface shear stress corresponding to the load increment from $M_p$ to $M_{ab}$ can be easily calculated, compares well to more complex analyses and is useful in design.
3. The scope of the proposed analytical was examined, as it relates to side-plating, wrapping around soffits, multiple layers of reinforcing, configuration of internal steel, and FRP extension to supports. Several beams with FRP wrapping and debonding mitigation were analyzed using the model, which was found to be conservative in every case.

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REFERENCES


