Mitigation of Concrete Cover Separation in Concrete Beams Strengthened with Fiber-reinforced Polymer Composites

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Abstract. Strengthening of reinforced concrete (RC) beams using fiber reinforced polymers (FRP) is an effective method that is gaining more acceptance worldwide. However, failure of the strengthened beams at the plate-end region by concrete cover separation (CCS) compromises the effectiveness of such a strengthening system. Among available anchorage systems to remedy such problem, the use of FRP U-wraps at the end region of the externally bonded FRP has the potential to prevent CCS failure. Most studies in the literature suffer from poor design of test specimens and the use of random amounts of FRP anchorage. This resulted in nonconclusive findings and contradicting results on the real effectiveness of FRP U-wraps as an anchorage to prevent plate-end debonding failure. That in turn has been reflected in the lack of reliable design methods for the design of FRP U-wrap anchorage. In this paper, the effect of FRP U-wraps on the mitigation of CCS failure in FRP-strengthened RC beams was experimentally examined and quantified. The study included full-scale RC beams strengthened in flexure using carbon FRP (CFRP) sheets and anchored at the plate-end region using vertical CFRP U-wraps. The test variable was the amount of CFRP U-wrap that is required to achieve complete mitigation of CCS. The test results indicated the effectiveness of FRP U-wraps on mitigating CCS failure once the required area of CFRP U-wrap was placed. Relevant design models were presented and assessed against the experimental results of the current study. The comparisons indicated inaccurate predictions for the area of FRP U-wrap that is required to prevent CCS. This sheds the light on the need for developing a more reliable and accurate method for the design of an end anchorage to prevent CCS failure.

1. Introduction

The application of externally bonded (EB) fiber reinforced polymer (FRP) for strengthening reinforced concrete (RC) beams is widely accepted in practice. This is attributed to the excellent characteristics of FRP materials that include high tensile strength, lightweight, corrosion resistance, and ease of installation. Numerous research work has been conducted to investigate the flexural performance of RC beams strengthened with EB FRP composites [1–7]. These studies have led to increase in the knowledge of the structural behavior of FRP-strengthened RC beams. However, premature debonding of the EB FRP from the concrete substrate is a serious drawback that compromises the utilization of the FRP composites. Such debonding failure could take place either at the middle of a simply supported beam, known as intermediate crack (IC) debonding, or at the end of the EB FRP reinforcement, plate end (PE)
debonding. Debonding at the plate end can occur either by concrete cover separation (CCS) or by interfacial (PEI) debonding. CCS is the most common type of PE debonding failure as reported in the literature [8,9]. The utilization of FRP tensile properties in FRP-strengthened beams experiencing PE debonding is considerably below the FRP rupture strain. In addition, PE debonding is very brittle as it occurs mostly before the yielding of tension steel. This necessitates the need for an anchorage of the EB FRP reinforcement at their ends to prevent the occurrence of such a brittle failure and enhance the efficiency of the FRP strengthening system [8].

Different anchorage systems for EB FRP have been developed and reported in the literature [8,10,11]. Unlike other anchorage methods, anchorage by means of FRP U-wraps does not require predrilling and do not suffer from corrosion. A review of different anchorage systems used in FRP-strengthening was conducted by Kalfat et al. [8], which indicated higher anchorage efficiency of FRP U-wraps as reported by Pham and Al-Mahaidi [12]. The use of FRP U-wraps has been commonly recommended by design guidelines [13-16] to reduce the risk and brittleness of PE debonding. Existing experimental studies [e.g. 10,12,17,18] have confirmed the effectiveness of FRP U-wrap anchorage in mitigating PE debonding failure. However, the findings from most available studies in the literature on the extent of improvement due to the use of FRP U-wraps are not clear and need further investigations. Moreover, lack of reliable design rules prevents the widespread use of FRP U-wrap anchorage. Therefore, this paper presents a systematic experimental study that quantifies the effect of FRP U-wrap as an end anchorage on the mitigation of PE debonding failure in FRP-strengthened beams.

2. Experimental study
This investigation focuses on evaluating the minimum amount of vertical FRP U-wrap that is required to prevent PE debonding failure. This amount is to be identified in terms of FRP U-wrap width.

2.1. Test beams
A total of five RC beams were constructed and tested in this study. The beams had the same geometry, with an overall depth of 400 mm, width of 200 mm, and a total length of 3400 mm as shown in Figure 1. One of the test specimens served as a reference beam, which was strengthened with EB carbon FRP (CFRP) laminate without an end anchorage. The rest of the test beams were strengthened with similar EB CFRP sheets and anchored with different amounts of FRP U-wraps at the FRP plate-end region. Table 1 shows the test matrix and lists details of the FRP U-wraps used as an end anchorage for the test beams. The test beams were designed carefully to ensure that the strengthened reference specimen (without FRP U-wraps) would fail by CCS. Furthermore, a large difference between the strengths controlled by CCS and other failure modes, mostly IC debonding, was intended in the design of the test specimens. The anchored beams contained one layer of CFRP U-wrap at the plate end. The thickness of U-wrap of this study was kept constant while the area was varied in terms of varying the width of the U-wrap as shown in Table 1.

The beams in this study were internally reinforced with two 16 mm-diameter tension steel bars and two 10 mm-diameter compression steel bars. Also, the beams were strengthened in flexure with two layers of CFRP sheets externally bonded to the tension face of the beam as wet lay-up system. Both the EB CFRP laminates and the FRP U-wraps were formed from plies of the same CFRP, with a dry ply nominal thickness of 0.41 mm. The EB CFRP laminates had the same nominal dimensions of 150 mm wide, 0.82 mm thick (i.e. two plies x 0.41 mm), and 2400 mm long for all the beams as shown in Figure 1. The FRP U-wrap was also formed in a wet layup process. To eliminate stress concentration and local failure at the bottom of FRP U-wrap, the bottom corners of the beams were rounded at a radius of 25 mm. The installation of the FRP strengthening and anchorage systems followed the recommendations provided by the manufacturer. Figure 2 shows schematically the layout of the anchored beams with FRP U-wraps.

Each anchored beam is identified with an acronym that consists of letter-number sets, except for the control specimen which was named reference beam (RB). The first symbol starts with the letter ‘U’ that stands for U-wrap anchorage, followed by a number 90 to denote that FRP U-wraps were perpendicular
to the longitudinal axis of the beam. The second symbol starts with the letter ‘W’ that stands for the width of the FRP U-wrap, followed by a number to denote the value of that width. For example, specimen U90-W100 is a beam anchored with vertical FRP U-wrap having a width of 100 mm.

![Figure 1. Geometry and reinforcement details of the test beams.](image1)

![Figure 2. Layout of the anchored beams with FRP U-wraps.](image2)

### Table 1. Details of the test beams.

| Beam     | FRP U-wrap width (mm) | FRP U-wrap thickness (mm) | End anchorage |
|----------|------------------------|---------------------------|--------------|
| RB       | -                      | -                         | No           |
| U90-W100 | 100                    | 1×0.41                    | Yes          |
| U90-W150 | 150                    | 1×0.41                    | Yes          |
| U90-W200 | 200                    | 1×0.41                    | Yes          |
| U90-W300 | 300                    | 1×0.41                    | Yes          |

2.2. Material properties
The test beams were constructed using a ready-mixed concrete provided by a local supplier. The concrete compressive strength of concrete was determined based on testing standard cylinders at the time of beam testing. The average compressive strength obtained from cylinder testing was 35 MPa.

The steel reinforcements used in the longitudinal and transverse directions were deformed steel bars. The stirrups were also deformed steel bars having a diameter of 10 mm and spaced at 150 mm. The tensile characteristics of the reinforcing bars were determined based on tensile tests carried out on representative specimens, as given in Table 2. The CFRP sheets used in flexural strengthening of the
beams or the U-wrap anchorage are Tyfo SCH-41. The sheets had a thickness of 0.41 mm and were bonded to the concrete surface using a two-component epoxy matrix, known commercially as Tyfo S Epoxy. The mechanical properties of the CFRP fabric and epoxy adhesive are given in Table 2 as provided by the manufacturer.

Table 2. Properties of steel reinforcement and FRP composite.

| Materials     | Yield stress (MPa) | Ultimate tensile stress (MPa) | Elastic modulus (MPa) | Ultimate strain (mm/mm) |
|---------------|--------------------|-------------------------------|-----------------------|--------------------------|
| Steel (16 mm) | 550                | 680                           | 205                   | -                        |
| Steel (10 mm) | 515                | 540                           | 200                   | -                        |
| CFRP sheet    | -                  | 3790                          | 230                   | 0.0165                   |
| Adhesive      | -                  | 72.4                          | 3.18                  | 0.05                     |

2.3. Test set-up
The beams were simply supported over a clear span of 3000 mm, and tested in four-point bending with a constant-moment region of 1200 mm, as shown in Figure 1. The shear span/depth ratio for the beams was kept constant at 2.56. Strain gauges were attached to the steel reinforcement and the top surface of concrete at mid-span to monitor the behavior of the test beams. Also, strains in the EB CFRP laminates and CFRP U-wraps were measured using strain gauges bonded at key locations. The deflection at mid-span was measured using two linear variable differential transducers (LVDTs) installed across the width of the beam. Also, one LVDT was installed under each point load to monitor the deflection profile of the beam. The load was monotonically applied using an MTS actuator at a stroke-controlled rate of 0.5 mm/min. The applied load, displacements, and strain readings were electronically recorded during the test using an electronic data acquisition system.

3. Test results
A summary of the experimental results for the test beams is given in Table 3. These key results include the load and corresponding mid-span deflection at debonding failure. It also shows the maximum strains in the EB CFRP, tension steel, CFRP U-wrap (measured at the level of tension steel), and concrete strain (measured at the compression surface of the beam mid-span) at beam failure. Also, the increase in the ultimate capacity of the anchored beams over that of the reference beam (strengthened without an end anchorage) is shown in Table 3 along with the mode of failure for the tested beams.

Table 3. Experimental results for the test beams.

| Beam     | Failure load (kN) | Ultimate deflection (mm) | Increase in capacity | Concrete strain | Steel strain | CFRP strain | CFRP U-wrap Strain a | Mode of failure b |
|----------|-------------------|--------------------------|----------------------|-----------------|--------------|--------------|----------------------|------------------|
| RB       | 216.7             | 13.3                     | -                    | 1150            | 2750         | 3080         | -                    | CCS              |
| U90-W100 | 263.8             | 18.7                     | 21.7%                | 1970            | 3420         | 4680         | 2360                 | CCS              |
| U90-W150 | 262.1             | 18.5                     | 20.9%                | 1380            | 4700         | 4540         | 2260                 | CCS              |
| U90-W200 | 268.8             | 17.2                     | 24.0%                | 1520            | 3740         | 4900         | 1500                 | ICD              |
| U90-W300 | 272.2             | 21.3                     | 25.6%                | 1210            | 5250         | 5100         | 1390                 | ICD              |

a Measured in CFRP U-wrap at the level of tension steel.
b CCS= Concrete cover separation; ICD= Intermediate crack debonding.
3.1. Ultimate capacity and mode of failure

The crack patterns at failure are shown in Figure 3 for the test beams. The reference beam (RB) failed by concrete cover separation (CCS) at an ultimate load of 216.7 kN. At a load of 135.0 kN inclined cracks started to form in the shear span and at the vicinity of the CFRP plate end. Prior to CCS failure, a horizontal crack started to form near the CFRP plate end and then propagated towards the middle of the beam along the level of tension steel bars. At an ultimate load of 216.7 kN, part of the concrete cover separated from the beam indicating the failure of the strengthened beam as shown in Figure 3. For the anchored beams, the use of FRP U-wrap at the plate end region did not cause a change in the crack spacing, as shown in Figure 3.

![Figure 3. Crack patterns at failure for the test beams.](image)

The effects of the amount of FRP U-wrap on the ultimate capacity and mode of failure is discussed herein. Figure 4 plots the failure load against the width of FRP U-wrap for the test beams. The percentage increase in the ultimate capacity of the anchored beams with respect to the reference beam was also given in the figure. Figure 4 and Table 3 show an increase in the failure load with increased width (and subsequently area) of the FRP U-wrap. Compared to the reference beam, an increase in the load at failure was found to be 21.7% and 20.9% for the beams anchored with FRP U-wraps having a width of 100 mm and 150 mm. In these beams, the width of used U-wrap was not sufficient to successfully prevent CCS failure. The beams failed by CCS starting at the inner side of the FRP U-jacket as shown in Figure 3. The debonding failure was associated with the rupture of FRP U-wrap at the beam bottom. This is an indicator that the used end anchorage in each of those beams was not sufficient to suppress the interfacial shear and normal stresses that caused CCS failure. The test results also showed that CCS was successfully prevented in beams U90-W200 and U90-W300. The two beams were end-anchored with sufficient areas of FRP U-wraps of 200 mm and 300 mm in width, respectively, with a corresponding increase in the ultimate load of 24.0% and 25.6% over that of the reference beam. The failure in these anchored beams has shifted to IC debonding that initiated at the middle of the beam. These test results indicate that the minimum FRP U-wrap width (and subsequently area) that is required to prevent CCS
was identified in beam U90-W200. On the other hand, using an end-anchorage of a larger width than that required to prevent CCS caused a marginal increase in the ultimate load at IC debonding failure as observed for beam U90-W300 of 300 mm U-wrap width.

3.2. Load-deflection relationships

The applied load versus mid-span deflection curves for the tested beams are shown in Figure 5. The reference beam RB exhibited a bilinear deflection relationship without showing a yielding plateau indicating a brittle performance. Similar behavior was also observed for the anchored beams with larger values of deflection at failure compared with the reference beam.

3.3. Strains in concrete and reinforcements

The reference beam (RB) failed by CCS at a small ultimate load with a low utilization of the constituent materials. The measured compressive strain of concrete at failure $\varepsilon_c$ was 1150 $\mu$e. On the other hand, using FRP U-wrap anchorage at the CFRP plate ends caused increments in the utilization of the constituent materials and thus the load at failure. The measured compressive strain of concrete at failure for the anchored beams was in the range 1210–1970 $\mu$e as given in Table 3. However, the compressive strength of concrete was not fully utilized because the failure still occurred either by CCS or IC debonding and none of the test beams failed by crushing of concrete.
The experimental results indicated a low utilization of the tensile strain capacity of tension steel reinforcement. The strengthened beam without end anchorage (RB) failed by CCS immediately after the yielding of tension steel. More utilization of tension steel was observed due to anchoring the test beams as indicated by the measured strains in tension steel at failure which ranged from 3420 µε to 5250 µε as shown in Table 3. The maximum strains developed in EB CFRP laminates under loading were observed in the constant moment region. Table 3 gives the strain in the laminates at failure as the average of the strains measured in that region. The maximum measured CFRP strain a failure for the reference beam was found to be 3080 µε which represents 18.7% of CFRP rupture strain. The anchored beams showed increase in the strain developed in CFRP laminates at failure as given in Table 3. In case of the beams anchored with U-wraps in which CCS was not prevented, the maximum measured strains in CFRP laminates were in the range of 4540–4680 µε, which represent 27.5–28.4% of CFRP rupture strain (with an increment in CFRP strains of 47.7%–51.9% over that of the reference beam RB). On the other hand, more utilization of the CFRP strain was observed for the beams with U-wrap anchorage in which CCS was successfully prevented and failure shifted to IC debonding. In these beams, the maximum measured strains in CFRP laminates ranged from 4900–5100 µε, which represent 29.7–30.9% of CFRP rupture strain (with increases of 59.1%–65.6% over that of the reference beam).

The strain distributions along the height of the FRP U-wrap at failure were plotted in Figure 6 for the beams anchored with FRP U-wrap at the plate end. The strain distribution for the beams failed in CCS was given in Figure 6a and that for the beams failed in ICD was given in Figure 6b.

![Figure 6. Strain distribution along FRP U-wraps at failure: (a) Beams with FRP U-wraps failed by CCS; (b) Beams with FRP U-wraps failed by ICD.](image-url)
Figure 6a shows that the maximum strain developed in the FRP U-wrap was at a level of 50 mm from the beam soffit which represents the level of tension steel bars. This may be expected as CCS generally occurs due to the horizontal splitting crack developed at this level. The measured strains at this level for all beams in the figure were larger than 2000 με. Moreover, the difference in the FRP U-wrap strains at the level of tension steel bars among the two beams was small. This indicates that a wider FRP U-wrap was required to provide a larger force for restraining the widening of the cracks at the plate end region and holding the longitudinal CFRP in place. Figure 6b indicates that the strains measured in the FRP U-wraps at the level of tension steel bars for the beams failed in IC debonding were found to be smaller than those observed in case of beams in which CCS was not prevented (Figure 6a). This can be also observed from the strain values given in Table 3. This indicates that the used U-wraps in these beams were sufficient to prevent CCS failure. The figure also shows that the measured U-wraps strain at the level of tension steel bars appeared to decrease with increasing the area of FRP U-wrap. This indicates that using more area of FRP U-wrap over the minimum required to prevent CCS was not of great benefit because the increased area of U-wrap provided insignificant increases in the ultimate capacities at IC debonding failure.

4. Comparison of test results with existing design models

There are limited provisions for the design of FRP U-wrap anchorage in FRP-strengthened RC beams. The design provisions recommended by the ACI 440 Committee [13], the Concrete Society Report [15], and the Chinese National Standard [14] are presented in Table 4.

| Table 4. Design equations for FRP U-wrap anchorage to prevent PE debonding failure. |
|-----------------------------------------------|
| **Reference** | **Design code equation** |
| ACI 440.2R [13] | \[ A_{fw} = A_{f} f_{fu} / \left( E_{frw} k_{v} \varepsilon_{frw,S} \right) \] where the factor \( k_{v} \) is a bond-reduction coefficient determined by: \[ k_{v} = \left( k_{1} k_{2} L_{c} / \left( 11900 \varepsilon_{frw,S} \right) \leq 0.75 \] The active bond length \( L_{c} \) can be computed as follows: \[ L_{c} = 23300 / \left( n_{frw} t_{frw} E_{frw} \right)^{0.58} \] The two factors, \( k_{1} \) and \( k_{2} \), account for the concrete strength and the type of wrapping scheme used, respectively. The calculations of these two factors are given by: \( k_{1} = \left( f'_{c} / 27 \right)^{2/3} \), \( k_{2} = \left( d_{fw} - L_{c} \right) / d_{fw} \) |
| Concrete Society [15] | \[ A_{fw} = A_{f} f_{fu} / \left( E_{frw} \varepsilon_{frw,e} \right) \] where \( \varepsilon_{frw,e} \) is the effective strain in the FRP U-wrap given by: \[ \varepsilon_{frw,e} = 0.5 \sqrt{f_{ct} / \left( n_{frw} t_{frw} E_{frw} \right)} \leq 0.004 \] where \( f_{ct} \) is the tensile strength of concrete and can be related to the cylinder compressive strength of concrete using the following equation CEB FIB [19]: \[ f_{ct} = 1.4 \left( f'_{c} - 8 / 10 \right)^{2} \] |
| Chinese National Standard [14] | The area of each FRP U-wrap should satisfy the following:

For wet lay-up FRP soffit plate: \( b_{fw} \geq \frac{3}{2} b_{s} \) and \( n_{frw} t_{frw} \geq \frac{1}{2} n_{f} t_{f} \)

Pultruded FRP soffit plate: \( b_{fw} \geq 100 \text{mm} \) and \( A_{fw} \geq \frac{1}{4} A_{f} \) |

The above provisions and models were used for calculating the FRP U-wrap area required to prevent CCS for the test beams of the current study. Figure 7 was plotted to compare the predictions of the available design provisions for the test beams, while Table 5 also shows this comparison. Figure 7 and Table 5 show that the design equations given by the ACI guidelines [13] and the Concrete Society [15] indicate the need for excessive U-wrap areas of 4.46 and 4.61 times that used experimentally in beam U90-W200, respectively. By contrast, the Chinese National Standard [14] calculates the need for a U-
wrap area of only 75% of that used in beam U90-W200, which is not adequate to prevent CCS. The comparisons presented above indicate that the available design methods give inaccurate predictions for the area of FRP U-wrap required to prevent CCS. This shows the need for a reliable design method for FRP U-wrap anchorage.

**Figure 7.** Comparison of available FRP U-wrap design equations with test beams.

**Table 5.** Comparison of calculated FRP U-wrap area with experimental values for the test beams.

| Beam   | Prevention of CCS | $A_{fw,exp}$ (mm$^2$) | ACI 440.2R [13] | TR55 [15] | GB-50608 [14] |
|--------|-------------------|------------------------|-----------------|----------|---------------|
| U90-W100 | NO               | 82.0                   | 8.92            | 9.21     | 1.50          |
| U90-W150 | NO               | 123.0                  | 5.95            | 6.14     | 1.00          |
| U90-W200 | YES              | 164.0                  | 4.46            | 4.61     | 0.75          |
| U90-W300 | YES              | 246.0                  | 2.97            | 3.07     | 0.50          |

5. Conclusions
This paper presented experimental investigation on the use of FRP U-wrap anchorage for flexural FRP reinforcement in RC beams. The study quantified the minimum area of FRP U-wrap required to prevent PE debonding. Based on the results of this investigation, the following conclusions can be made:

- Increasing the width of FRP U-wraps caused an increment in both the ultimate load and deflection of the strengthened beam. In the test beams anchored with an inadequate area of U-wrap (a U-wrap width of 100 and 150 mm), the beam still failed by concrete cover separation that initiated at the inner side of the FRP U-wrap.

- The minimum required area of FRP U-wraps to safely guard against concrete cover separation was identified in the test beam anchored by a U-wrap of 200 mm width, with an increment in the failure load of 24.0% compared to the reference beam.

- Using more FRP U-wrap area than that required to prevent PE debonding showed a marginal increment in the load at failure and contributed to a gradual failure by IC debonding upon the successful prevention of PE debonding.

- A comparison of available design code equations was conducted with the experimental results. The comparison indicated excessively high area of FRP U-wrap given by these equations compared to the experimentally used areas. This indicates the need for a more accurate and reliable design method for the required FRP U-wrap area to prevent plate-end debonding failure.
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