Experimental and theoretical analysis of CFRP reinforced concrete beam anchored by CF anchors

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Abstract. Carbon fiber-reinforced polymer (CFRP) sheets is used to strengthen RC members widely. However, in the actual projects, premature separation of the CFRP sheet from the concrete surface is observed frequently, key issue in CFRP application is the performance of the connection between the CFRP sheet and the peripheral RC members. This paper presented the results of an experimental study to investigate mechanical properties of carbon fiber (CF) anchor dowels, a new CFRP anchoring method, and based on the experimental results, obtained the bearing capacity calculation formula of concrete beam strengthened with CFRP anchoring by carbon fiber anchor dowels.

1. Introduction

The carbon fiber reinforced polymer (CFRP) has a high tensile strength, light weight, strong corrosion resistance, simple and easy to operate compared with other reinforcement methods, and the construction process does not affect the duty of the structure. There have been quite a lot of studies report that externally bonded CFRP could effectively improve carrying capacity and change failure mode of RC members\cite{1-3}. In order to improve bearing capacity of RC beams, the bonding strengthen between CFRP and RC members must be strong enough to make sure the full use of tensile strength of CFRP\cite{4-6}. A study of behaviour of reinforced concrete beams strengthened with FRP found that CFRP sheets debonded on average when suffering 50\% of their tensile capacity\cite{7}. So premature debonding failure due to separation of laminates caused by excessive stress concentration at the end of laminate had been agreed to be the main problem. The risk of Debonding at laminate ends could be minimized using appropriate end anchors. Therefore, in order to more fully utilize the tensile capacity of the FRP sheet, carbon fiber (CF) anchors which were originally developed by the Japanese Shimizu Corporation were used as a new way to anchor FRP sheets.

2. Experimental study

2.1. Test program

In this study, two RC beams had been tested to failure by four-point bending method. Specimens had cross-sections of 200 mm width by 270 mm height and length of 2,260 mm. One of beams strengthened without anchoring as a control beam named L0, and the other beam named L1 was strengthened by
CFRP sheet and anchored by CF anchors at two ends. Beams strengthened with two steel bars of 12 mm diameter with a clear cover of 35 mm. In the test program, all the beam specimens were tested in four-point bending with an effective span of 1,960 mm and a shear span of 880 mm, as showed in Figure 1. Load was applied in displacement rate of 1.5 mm/min. the beam specimens were instrumented with linear variable differential transformer (LVDT) and strain gauges installed to measure vertical strain at the midspan of beams.

![Figure 1. Load test setup](image)

2.2. Settings of CF anchors
CF anchors were made as follow procedures: firstly cut a piece of CFRP sheets which as same as CFRP sheet material bonded in the beam span. Then CF anchors were saturated with epoxy and installed immediately into a predrilled and carefully cleaned holes in the beam face after CFRP sheet for strengthening was bonded. The ends of the CF anchors were fanned out over the CFRP sheet, as shown in Fig. 2.

![Figure 2. The mounting process of CF anchors](image)

2.3. Experimental phenomena and results analysis
2.3.1. Cracks and failure mode. The cracks of control beam L0 was wider than strengthened beam L1 while the amount of cracks was less than L1’s. Beam L0 shows a larger deflection than beam L1’s when both of them was failed. It can be deduced that failure mode of beam L1 was the crush of concrete, which means beam L1 had a greater stiffness. The failure mode of beam that was not strengthened was flexural failure at the middle span, as shown in Fig. 3.

![Figure 3. Fracture morphology of test beams](image)

2.3.2. Distribution of strain. The readings of strain gauges mounted in the gauging points as shown in Fig.1 are shown in Fig. 4.

There was an obvious linear distribution of strain along the altitude-direction of the section in the middle span in the elastic stage, which basically agreeing with the plane section assumption. The location of neutral axis of beam L0 was close to the center of the specimen, while the location of neutral axis of beam L1 was lower than L0’s because of the CFRP’s high elasticity modulus.
3. **Theoretical analysis**

The calculation model of the ultimate bearing capacity of the beam strengthened with CFRP sheets was deduced based on failure mode which is concrete crushing. Ultimate compressive strength of concrete was used as a control parameter in the progress of deducing.

3.1. **Basic assumptions**

a) Hypothesis of plain section which had proved in the beam span strain analysis.

b) CFRP only bears axial tension and do not bear bending moments.

c) Take no account of the concrete resisting tensile effect.

d) Using elastic-all plastic type curve to describe the change of steel bar, regardless of the strengthening phase; Using linear elastic stress - strain curve to describe the change of carbon fiber, magnitude of stress is equal to the strain of CFRP multiplied by the modulus.

3.2. **Ultimate bearing capacity under beams destroying**

The sectional strain and stress distribution along the section height for CFRP strengthened beams anchored by CF anchors was given in Fig.5.

![Figure 5. Sectional strain and stress distribution along the beam height](image)

Considering compressive strain was positive, the stress and the steel strain of CFRP sheet was negative. According to the assumption of plane, the CFRP strain $\varepsilon_f$ and the steel bars strain $\varepsilon_s$ could be calculated as follows:

$$
\varepsilon_f = \varepsilon_{cf} \frac{x - e_i}{x}, \quad \varepsilon_s = \varepsilon_{cf} \frac{x - d_s}{x}.
$$

(1)

Where $x$ was the distance between concrete compression zone edge and the neutral axis, $d_s$ was the distance between concrete compression zone edge and centroid of steel bars, $d_f$ was the distance between concrete compression zone edge and CFRP’s centroid. According to The Chinese Concrete Design Specification (GB50010-2012), the concrete strain could be calculated:

$$
0 \leq \varepsilon_c \leq \varepsilon_{c0}, \quad \sigma_c = f_c \left[1 - \left(1 - \frac{\varepsilon_c}{\varepsilon_{c0}}\right)^n\right], \quad n = 2 - \frac{1}{60} (f_{cu,h} - 50).
$$

(2)

$$
\varepsilon_0 \leq \varepsilon_c \leq \varepsilon_{c1}, \quad \sigma_c = f_c \varepsilon_c, \quad \varepsilon_{c1} = 0.002 + 0.5(f_{cu,h} - 50) \times 10^{-3}, \quad \varepsilon_{c0} = 0.003 - 0.5(f_{cu,h} - 50) \times 10^{-3}.
$$

(3)
Where $\sigma_c$ was concrete compressive stress, $f_c$ was the design value of concrete compressive strength, $\varepsilon_0$ was the concrete compressive strain when the concrete compressive stress value was equal to $f_c$, and when the calculated value of $\varepsilon_c$ was less than 0.0002, take it as 0.0002. $f_{cu,k}$ was the compressive strength standard values of concrete cube. $n$ was a given coefficient, when the calculating value of $n$ was greater than 2.0, take it as 2.0. In this paper, the concrete strength level was 30 MPa, so writer set the value of $\varepsilon_{cu}$ as 0.0033, $n$ as 2.0. Concrete resultant compressive stress $C$ was calculated as follows:

$$C = k_{i}f_{i}b_{x}, \quad k_{i} = \frac{\int_{0}^{\varepsilon_{0}} \sigma_{ci} d\varepsilon_{ci}}{f_{eff}}.$$  \hspace{1cm} (4)

$$0 \leq \varepsilon_{c} \leq \varepsilon_{0}, k_{i} = \left( \frac{\varepsilon_{0}}{\varepsilon_{c} - 3\varepsilon_{0}} \right), \quad \frac{\varepsilon_{0}}{\varepsilon_{c}} \leq 0.0033, \quad k_{1} = 1 - \frac{\varepsilon_{0}}{3\varepsilon_{c}}. \hspace{1cm} (5)$$

Where $b$ was the width of the beam, $k_{i}$ was the average stress coefficient. The static equilibrium equation is:

$$k_{i}f_{i}b_{x} + \sum_{i=1}^{n} \sigma_{u}A_{u} + \sigma_{f}A_{f} = 0. \hspace{1cm} (6)$$

Where $\sigma_{u}$ was the stress of steel bars, $\sigma_{f}$ is the stress of CFRP, $A_{u}$, $A_{f}$ is a total area of the ith layer and the sectional area of CFRP respectively. $n$ is the total number of steel bars layers. The $\sigma_{u}$ and $\sigma_{f}$ were calculated:

$$k_{i}f_{i}b_{x}\sigma_{u} = E_{s}\varepsilon_{u}, \quad k_{i}f_{i}b_{x}\sigma_{f} = E_{t}\varepsilon_{f}.$$  \hspace{1cm} (7)

Where $E_{s}$, $E_{t}$ was the elasticity modulus of steel bar and CFRP respectively. $f_{f}$ was the tensile strength of CFRP, while $f_{f}$ was the yield strength of steel bar. $\gamma_{s}$, $\gamma_{f}$ was the material subentry coefficient of steel bar and CFRP, and both positive.

If $D$ was the distance between the resultant compressive stress and the edge of the concrete compressive zone, and $x$ was the height of compression zone, then:

$$D = k_{2}x, \quad k_{2} = 1 - \frac{\int_{0}^{\varepsilon_{0}} \sigma_{ci} d\varepsilon_{ci}}{\int_{0}^{\varepsilon_{0}} \sigma_{ci} d\varepsilon_{ci}}.$$  \hspace{1cm} (8)

CFRP achieving ultimate tensile strain or concrete compressive strain reaching to 0.0033 (the ultimate compressive strain) were failure signs of RC beam. When the failure mode was the concrete crush ($\varepsilon_{cu}$ = 0.0033), from (1) CFRP sheet’s stress distribution $\varepsilon_{f}$ could be got, and according to (5), the average stress coefficient $k_{1}$ could be calculated.

Based on (1) to (8), the neutral axis depth $x$ could be caudated:

$$\varepsilon_{x} = \frac{f_{f}}{\gamma_{f}E_{f}}, \quad x = - \frac{\sum_{i=1}^{n} E_{f}\varepsilon_{u}A_{u} + E_{f}\varepsilon_{f}A_{f}}{\left(1 - \frac{\varepsilon_{0}}{0.00999}\right) f_{f}b}.$$  \hspace{1cm} (9)

$$\varepsilon_{x} \geq \frac{f_{f}}{\gamma_{f}E_{f}}, \quad x = - \frac{\sum_{i=1}^{n} E_{f}\varepsilon_{u}A_{u} + E_{f}\varepsilon_{f}A_{f}}{\left(1 - \frac{\varepsilon_{0}}{0.00999}\right) f_{f}b}.$$  \hspace{1cm} (10)

Then the ultimate bending moment of CFRP reinforced concrete beams was calculated:

$$M_{u} = k_{1}f_{f}b_{x}\left(\frac{h}{2} - k_{2}x\right) + \sum_{i=1}^{n} \sigma_{u}A_{u}\left(\frac{h}{2} - d_{i}\right) + \sigma_{f}A_{f}\left(\frac{h}{2} - d_{f}\right).$$  \hspace{1cm} (11)

Where $h$ was the height of the concrete beam. The right three parts of formula were the contribution of concrete, steel and CFRP to the ultimate capacity.
4. The comparison between the theoretical and the experimental value

The theoretical and the experimental ultimate bearing capacity of experiment beams was shew in Fig.6.

The ultimate bearing capacity of the beam’s theoretical value was 45.8kN, which was only 3.4% higher compared to experimental value of 44.3kN. The error may be caused by the bond between concrete face and CFRP sheet. The two kinds of values were very close this indicated that the theoretical formula was corrected according to the hypothesis of the test.

![Figure 6. The ultimate bearing capacity value of experiment beams](image)

5. Conclusion

According to the experiment, the failure model of CFRP strengthened beam that was anchored by CF anchors was crash of concrete, while failure model of beam that was not strengthened was flexural failure at the middle of beam. In the process of loading, the measured beam strain along the section height was the linear distribution and basically accord with flat section assumption. Based on the test results, the formula of the bearing capacity of the beam anchored by CF anchors was established in this paper. Compared the experiment and the theoretical analysis, we concluded that the theoretical calculated value shew a good agreement with experimental data, which indicated that the theoretical formula was relatively reasonable.

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