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Concrete Filled Carbon FRP Tube (CFRP-CFFT) Columns with and without CFRP Reinforcing bars: Axial-Flexural Interactions

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Abstract

The axial and flexural behaviors of Concrete Filled Carbon Fiber Reinforced Polymer Tube (CFRP-CFFT) columns have received significant research attention in the last two decades. One of the most attractive advantages of Carbon FRP (CFRP) tube is the high confinement which results in substantial increase in peak axial and flexural loads and deformations. Despite large research efforts, the behavior of CFRP-CFFT with and without CFRP reinforcing bars under different applied axial load eccentricity has not yet been adequately investigated. This study investigates the experimental and analytical axial-flexural ($P-M$) interactions of CFRP-CFFT columns with and without CFRP reinforcing bars. A total of 12 specimens of 204 – 205 mm outer diameter and 800 – 812 mm height were tested under concentric axial load, 25 mm and 50 mm eccentric axial loads and four-point load. The effectiveness of CFRP reinforcement (tube and bar) was observed to be reduced with the increase in the applied axial load eccentricity. Analytical $P-M$ interactions were constructed using available FRP confined concrete design codes which matched well with the experimental $P-M$ interactions. The parametric study showed that the actual confinement ratio, orientation of fibers and CFRP bar reinforcement ratio have significant influences on $P-M$ interactions of CFRP-CFFT specimens.

Keywords: A. Carbon fiber; B. Strength; C. Analytical Modeling; D. Mechanical Testing

1. Introduction

Steel bar Reinforced Concrete (RC) structures have been used for more than 100 years. The major factor limiting the design life of steel RC structures is the corrosion of reinforcing steel bar. When corroded, the steel bar loses strength and leads to deterioration in the strength and ductility of steel RC structural members. One of the solutions to reduce deterioration in the strength and ductility of steel RC structural members is to use Fiber Reinforced Polymer (FRP) reinforcement in lieu of steel reinforcement. FRP reinforcement has several advantages over steel reinforcement such as higher corrosion resistance, higher strength to weight ratio and superior durability in aggressive environment [1].

The Concrete Filled FRP Tube (CFFT) was introduced about two decades ago for efficient use of FRP reinforcement for new column construction [2]. The CFFT combines the advanced composite material (FRP tube) with the conventional material (concrete) to attain increased strength and ductility of column by restraining the lateral dilation of concrete. In CFFT, FRP tube serves as light-weight corrosion resistant permanent structural formwork and also serves as longitudinal and transverse reinforcements, depending on the orientation of fibers in FRP tube [3].

The axial and flexural behaviors of CFFT have received considerable research attention in the last two decades. Experimental investigations demonstrated that circular CFFT specimens tested under axial compressive loads exhibited increased confined concrete strength and ductility due to higher confinement provided by the FRP tube compared to the confinement provided by the steel helix in conventional steel bar RC specimens [1]; [3-14]. A limited number of experimental studies, however, investigated the flexural behavior of CFFT specimens. The experimental investigations reported that CFFT specimens exhibited higher flexural strength than equivalent steel RC specimens. The failure mode of CFFT specimens under flexural load was pseudo-ductile with significant warning before failure. Moreover, in CFFT, fibers in the longitudinal direction were effective in resisting flexural load whereas fibers in the circumferential direction were effective in confining the concrete and increasing the shear resistance of the concrete [15-18].

In recent years, FRP bar has been investigated as a viable alternative of steel bar in RC structural members in regions where corrosion of steel is a major concern. A number of research studies

investigated the axial compressive behavior of FRP bar reinforced concrete columns as an alternative of steel bar reinforced concrete columns. These research studies reported that FRP bar reinforced concrete columns exhibited lower axial load capacity compared to equivalent steel RC columns. The FRP bars were effective in resisting axial loads and hence the contribution of FRP bars should be adequately accounted for in the axial load capacity of FRP bar reinforced concrete columns [19-25]. The flexural behavior of FRP bar reinforced concrete beams was extensively investigated. The FRP bar reinforced concrete beams under flexural load exhibited similar ultimate flexural strength to the equivalent steel RC beams. However, the crack width and depth in FRP bar reinforced concrete beams were larger in steel bar reinforced concrete beams due to lower modulus of elasticity of FRP bars than the modulus of elasticity of steel bars [26-30].

Only a limited number of the studies investigated axial or axial-flexural behavior of Concrete Filled Glass FRP Tube (GFRP-CFFT) columns with reinforcing bars. These studies reported that confinement effectiveness of GFRP tubes significantly decreased with increased applied axial load eccentricity[13], [31]. Hadood *et al.* [32] investigated the axial-flexural behavior of GFRP bar reinforced concrete columns confined with GFRP helices. Hadood *et al.* [32] reported that GFRP helices prevented the buckling of GFRP bars and crushing of concrete core up to the failure of columns, and GFRP helices were efficient in confining the columns under varying applied axial load eccentricity. Saljoughian and Mostofinejad [33] reported that RC columns confined with intermittent CFRP sheets exhibited higher axial load carrying capacity and ductility under applied axial load eccentricity than unconfined RC columns. However, to the knowledge of the authors, no study investigated the axial-flexural ($P-M$) behavior of Concrete Filled Carbon FRP Tube (CFRP-CFFT) columns with and without Carbon FRP (CFRP) reinforcing bars. This study analytically investigates the axial-flexural interactions of CFRP-CFFT specimens with and without CFRP reinforcing bars.

2. Experimental Program

The experimental program presented herein is part of the ongoing research studies by the authors and research collaborators on the use of advanced composite materials in infrastructure. As part of the large experimental research program, 12 steel RC and CFRP-CFFT specimens of 204 – 205 mm outer

diameter and 800 – 812 mm height were tested under concentric axial load, 25 mm and 50 mm eccentric axial loads and four-point load. The details of the failure modes, load-deformation behavior of the tested specimens, illustration of test set-ups and test results of specimens were presented in Hadi *et al.* [1]. A brief description of the experimental procedure and results has been presented herein for completeness.

The specimens were cast and tested at the High Bay Laboratories, School of Civil, Mining and Environmental Engineering, University of Wollongong, Australia. The specimens were divided into three groups of four specimens. The first group, Group REF consisted of four steel bar Reinforced Concrete (RC) specimens. The second group, Group CT consisted of four CFRP-CFFT specimens. The third group, Group CTCR consisted of CFRP bar reinforced CFRP-CFFT specimens. From each group, one specimen was tested under concentric axial load, one under 25 mm eccentric axial load, one under 50 mm eccentric axial load and one under four-point load. The test matrix is presented in Table 1. The notations of the specimens comprised two parts. The first part indicates the type of specimen (REF, CT and CTCR). The second part indicates the load conditions (0, 25, 50 and B) where 0 indicates concentric axial load, 25 indicates 25 mm eccentric axial load, 50 indicates 50 mm eccentric axial load and B indicates four-point load.

In Group REF, six N12 (12 mm diameter deformed with 500 MPa nominal tensile strength) steel bars were used as longitudinal reinforcement and R10 (10 mm diameter plain bar with 250 MPa nominal tensile strength) steel helix of 165 mm outer diameter with 60 mm pitch was used as helical reinforcement. In Group REF, the concrete clear cover at top and bottom ends was 15 mm and at the side of specimen was 20 mm. Groups CT and CTCR consisted of 0.5 mm nominal thick CFRP tubes with outer layers of fibers oriented at $\pm 60^\circ$ to the longitudinal direction (skew fibers) and inner layers of fibers oriented at 90° to the longitudinal direction (circumferential fibers). The CFRP tubes consisted of 66% fibers oriented at $\pm 60^\circ$ to the longitudinal direction and 34% fibers oriented at 90° to the longitudinal direction. The CFRP tubes consisted of 37% resin and 63% fibers by volume [34]. The moduli of elasticity of CFRP tube in the circumferential and longitudinal directions were 54 GPa and 16.2 GPa, respectively [34]. The ultimate tensile strengths of CFRP tube in the circumferential and longitudinal directions were 1188 MPa and 142.6 MPa, respectively [34]. In Group CTCR, six CFRP bars of 15 mm nominal diameter were glued to the inner side of the CFRP tube (60° apart) along the circumference. The

CFRP bars comprised 55-60% carbon fibers and 40-45% vinyl ester resin by volume. The CFRP bars were manufactured by pultrusion [34]. For the specimens of Group CTCR, the concrete clear cover at the top and bottom ends was 15 mm. The specimens were cast with a ready mix concrete obtained from a local manufacturer. The specimens were cured by covering them with wet hessian rugs and plastic sheets for 28 days.

The average compressive cylinder strength of concrete on the 28th day tested according to AS 1012.9-1999 [35] was 37 MPa. Tensile testing of N12 and R10 steel bars was carried out according to AS 1391-2007 [36]. The nominal average tensile strengths of N12 and R10 steel bars were 600 MPa and 400 MPa, respectively. The cross-sectional area of CFRP bar was measured using immersion testing according to ISO 10461-1-15 [37]. The measured and manufacturer provided nominal cross-sectional areas of CFRP bar were similar (177 mm²). The CFRP bars were tested in tension according to ASTM D7205/D7205M-11 [38]. The modulus of elasticity of CFRP bar in tension was 89.4 GPa and the ultimate tensile strength of CFRP bar was 1157 MPa. The CFRP bars were tested in compression according to ASTM D695-10 [39]. The average modulus of elasticity of CFRP bar in compression was 49 GPa and the average ultimate compressive strength of CFRP bar was 596 MPa. The reduction factor (α), ratio of average ultimate compressive strength to average ultimate tensile strength of tested CFRP bars, was found to be 0.52.

2.1. Instrumentation and Test Procedures

The specimens of Group REF were instrumented at the mid-height with two strain gauges fixed on steel helix (180° apart) to measure the circumferential strains in steel helix and two strain gauges were fixed on two steel bars (180° apart) to measure longitudinal strains in steel bars. The specimens of Groups CT and CTCR were instrumented at the mid-height with two strain gauges fixed in the circumferential direction on the CFRP tube (180° apart) to measure the circumferential strains. In the specimens of Group CTCR, a pair of strain gauges was attached on two CFRP bars at the mid-height of the specimens (180° apart) to measure the axial strains in CFRP bars. A laser triangulation was fixed at the mid-height of specimens tested under eccentric axial loads to measure lateral deformations. A laser triangulation was also fixed at the midspan of specimens tested under four-point load to measure midspan deflections of

the specimens. All the specimens were externally instrumented with two Linear Variable Displacement Transducers (LVDTs) fixed diagonally (180° apart) in the test machine to measure axial deformations. All specimens were tested in the 5000 kN Denison Testing Machine. The specimens were preloaded to 100 kN and unloaded to 20 kN under a force controlled load application at a rate of 50 kN/min. Initial loading-unloading was carried out so that specimens placed in testing machine were aligned properly to the loading plates. Afterwards, testing was resumed under a displacement controlled load application at a rate of 0.3 – 0.5 mm/min until the failure of the specimen.

2.2. Experimental Axial Flexural Interactions

Experimental axial flexural ($P-M$) interactions of Groups REF, CT and CTCR under concentric axial load, 25 mm and 50 mm eccentric axial loads and four-point load have been presented in Fig. 1. The peak axial loads (P) and lateral deformation corresponding to the peak axial loads (δ) were selected to construct $P-M$ interactions. In the specimens of Group REF, the peak axial load represents the maximum axial load sustained by the gross concrete (cover and core) section. In the specimens of Groups CT and CTCR, the peak axial load represents the maximum axial load carried by the specimens before the rupture of CFRP tube. Bending moment (M) capacity of specimens tested as columns were calculated considering moments due to applied load eccentricity (e) and lateral deformation (δ) at peak axial load (Equation 1) and bending moment capacity of specimens tested as beams were calculated using Equation (2):

$$M = P(e + \delta) \quad (1)$$

$$M = \frac{Pl}{6} \quad (2)$$

where l is the span length of flexural test arrangement which was 705 mm.

The experimental load deformation curves of specimens tested under different loading conditions of Groups CT, CTCR and REF are presented in Fig. 2. Specimen CT-0 carried 15.8% larger axial load than Specimen REF-0 and Specimen CTCR-0 carried 43.7% larger axial load than Specimen REF-0. This is because the CFRP tube provides higher confinement than that of steel helix at the peak axial load. Also, the mid-height circumferential strains at the peak axial load on compression and tension sides in CFRP

tube were significantly higher than those in the steel helix (Table 2). This is because two-thirds of the total fibers in CFRP tube were oriented along the circumferential direction, which were effective in confining the concrete. Also, the CFRP tube provided continuous confinement to the concrete.

Specimen CT-25 carried 15% smaller axial load than Specimen REF-25 and Specimen CT-50 carried 17.7% smaller axial load than Specimen REF-50. Specimen CT-25 exhibited 10.8% smaller bending moment than Specimen REF-25 and Specimen CT-50 exhibited 16.8% smaller bending moment than Specimen REF-50. On the other hand, Specimen CTCR-25 carried 33.1% larger axial load than Specimen REF-25, and Specimen CTCR-50 carried 12.1% larger axial load than Specimen CT-50. Specimen CTCR-25 exhibited 75.4% larger bending moment than Specimen REF-25, and Specimen CTCR-50 exhibited 45.6% larger bending moment than Specimen REF-50. The increase in the applied axial load eccentricity resulted in larger reduction in the peak axial loads, axial deformations at peak axial load and corresponding bending moment in the specimens of Groups CT and CTCR than in the specimens of Group REF. This is attributed to the lower modulus of elasticity of the CFRP reinforcement than steel reinforcement. The increase in applied axial load eccentricity from 0 to 25 mm and 25 to 50 mm resulted in about 75% and 93%, respectively, reduction in the mid-height circumferential strains at peak axial load in CFRP tube. Moreover, the increase in applied axial load eccentricity resulted in larger reduction in axial strains at the mid-height in steel bars than CFRP bars which is attributed to the fact that CFRP bars have lower modulus of elasticity than steel bars hence CFRP bars developed higher strains than steel bars. The steel bars carried about 26.2%, 26.0% and 4.6% of the peak axial load carried by Specimens REF-0, REF-25 and REF-50, respectively. The CFRP bars carried about 12.3%, 15.6% and 15.7% of the peak axial load carried by Specimens CTCR-0, CTCR-25 and CTCR-50, respectively.

Specimen CT-B exhibited 59.3% smaller bending moment than Specimen REF-B. Specimen CTCR-B exhibited 2.6% smaller bending moment than Specimen REF-B. Specimens CT-B and CTCR-B carried lower peak flexural loads and corresponding midspan deflections than Specimen REF-B (Table 3). This was attributed to the fact that fibers oriented in the circumferential direction in the CFRP tube were ineffective in resisting flexural loads. However, the fibers oriented in the longitudinal direction in the CFRP tube were effective in resisting flexural loads as indicated by the longitudinal strains in CFRP

tubes at peak flexural load (Table 3). Moreover, the smooth CFRP bars slipped under the four-point load and hence the strain in CFRP bars at peak flexural load was significantly lower than that in steel bars.

It can be concluded that Specimens CT can serve as an alternative of Specimen REF only under concentric axial load as the increase in the applied axial load eccentricity resulted in larger reduction in peak axial load and corresponding mid-height circumferential strains in CFRP tube confined concrete specimens than in conventional steel bar reinforced concrete (REF) specimens. Specimens CTCR can serve as an alternative of Specimens REF under concentric and eccentric axial loads.

3. Development of Analytical Axial Flexural Interactions

Analytical axial flexural ($P-M$) interactions of Groups REF, CT and CTCR were developed using the Equivalent Rectangular Stress Block method satisfying the strain compatibility and force equilibrium conditions. The details of the modeling of Groups REF, CT and CTCR are presented below.

3.1. Modeling for Group REF

For specimens in Group REF, the concrete was modeled as unconfined concrete and the steel bar was modeled as an elastic-perfectly plastic material.

3.1.1. Modeling of Concrete in Group REF

The concrete in the specimens of Group REF was modeled as unconfined concrete by ignoring the confinement provided by steel helix at peak axial load. This approach is consistent with the ACI 318-11 [40] design guidelines for structural concrete, which ignores the contribution of steel helix confinement at peak axial load.

3.1.2. Modeling of Steel bars in Group REF

The steel bar in the specimens of Group REF was modeled as an elastic-perfectly plastic material. The axial stress in steel bar (f_s) at a given axial strain (ϵ_s) was determined as a function of modulus of elasticity of steel bar (E_s), as given in Equation (3):

$$f_s = E_s \epsilon_s \leq f_y \quad (3)$$

213 where f_y = yield strength of steel bar

214 **3.2. Modeling for Groups CT and CTCR**

215 For specimens in Groups CT and CTCR, the concrete was modeled as confined concrete. In Group
216 CTCR, the CFRP bar was modeled as a linear elastic material.

217 **3.2.1. Modeling of Concrete in Groups CT and CTCR**

218 The concrete in the specimens of Groups CT and CTCR was modeled as confined concrete using
219 available FRP confined concrete design codes i.e., ACI 440.2R-08 [41] and *fib* Bulletin 14 [42], and
220 Samaan *et al.* [43] stress-strain model for FRP confined concrete.

221 The ACI 440.2R-08 [41] has adopted the design oriented Lam and Teng [44] stress-strain model for the
222 FRP confined concrete subjected to combined axial load and bending forces. The ACI 440.2R-08 [41]
223 proposed Equations (4) and (5) to calculate the confined concrete strength (f'_{cc}) and the ultimate FRP
224 confined concrete strain (ϵ_{cu}), respectively.

$$f'_{cc} = f_{co} + \psi_f 3.3 f_{l,a} \quad (4)$$

$$\epsilon_{cu} = \epsilon_{co} \left(1.50 + 12 \frac{f_{l,a}}{f_{co}} \left(\frac{\epsilon_{rup}}{\epsilon_{co}} \right)^{0.45} \right) \epsilon_{cu} \leq 0.01 \quad (5)$$

225 where ψ_f is a reduction factor and is equal to 0.95, $f_{l,a}$ is the actual confinement pressure and ϵ_{rup} is the
226 circumferential rupture strain of fibers.

227 The *fib* Bulletin 14 [42] adopted the analysis oriented stress-strain model in Spoelstra and Monti [45]
228 without any modification. In *fib* Bulletin 14 [42], the ultimate FRP confined concrete strength (f'_{cc}) and
229 the ultimate FRP confined concrete strain (ϵ_{cu}) were calculated using Equations (6) and (7), respectively.

$$f'_{cc} = f_{co} \left[2.254 \sqrt{1 + 7.94 \frac{f_{l,a}}{f_{co}}} - 2 \frac{f_{l,a}}{f_{co}} - 1.254 \right] \quad (6)$$

$$\varepsilon_{cu} = \left[1 + 5 \left(\frac{f'_{cc}}{f_{co}} - 1 \right) \right] \quad (7)$$

For comparison purposes, the stress-strain model in Samaan *et al.* [43] was also selected to calculate the confined concrete strength (f'_{cc}) (Equations (8) and the ultimate FRP confined concrete strain (ε_{cu}) (Equation (9) of CFRP tube confined concrete specimens.

$$f'_{cc} = f_{co} + 6f_{l,a}^{0.7} \quad (8)$$

$$\varepsilon_{cu} = \frac{f'_{cc} - f_o}{E_2} \quad (9)$$

where

$$f_o = 0.872 f_{co} + 0.371 f_{l,a} + 6.258 \quad (10)$$

$$E_2 = 245.61 f_{co}^{0.2} + 1.3456 \frac{E_{FRP} t_{FRP}}{D} \quad (11)$$

3.2.2. Modeling of FRP bars in Group CTCR

For specimens of Group CTCR, the axial stress-axial strain behavior of CFRP bar was modeled as linear elastic till failure as in Deitz *et al.* [46]. The axial stress (f_{CFRP}) at any given axial strain (ε_{CFRP}) in CFRP bar was determined as a function of modulus of elasticity of CFRP bar in compression (E_{CFRP}) as given in Equation (12).

$$f_{CFRP} = E_{CFRP} \varepsilon_{CFRP} \quad (12)$$

4. Analytical Axial Flexural Interactions

Analytical axial flexural ($P-M$) interactions of steel RC (Group REF), CFRP-CFFT (Group CT) and CFRP bar reinforced CFRP-CFFT (Group CTCR) specimens were constructed using the equivalent rectangular stress block (hereafter, referred as stress block) method. The axial load capacity of specimens of Group REF under concentric axial load was calculated using Equation (13) and the axial load capacity of specimens of Groups CT and CTCR under concentric axial load was calculated using Equation (14).

$$P_n = 0.85 f_{co} (A_g - A_s) + f_y A_s \quad (13)$$

$$P_n = 0.85 f'_{cc} A_g + \alpha f_{fu} A_{CFRP} \quad (14)$$

where A_g is the gross sectional area of concrete, A_s is the area of steel bars, f_y is the yield strength of steel bar, α is a reduction factor to account for the lower compressive strength of CFRP bar than the tensile strength and is taken as 0.52 based on the compressive and tensile tests of CFRP bars as reported above in the experimental program, f_{fu} is the ultimate tensile strength of CFRP bar and A_{CFRP} is the area of longitudinal CFRP bars. The axial load and bending moments of Groups REF, CT and CTCR under eccentric axial loads and four-point load were calculated using the stress block method.

4.1. Stress Block Method

In the stress block method, a uniform stress distribution instead of nonlinear stress distribution above the neutral axis was considered to estimate the strength of concrete cross-section. An equivalent rectangular stress block of width $\alpha_2 f_{co}$ and depth γd_N was assumed to act over the effective circular concrete area (A_c) (Fig. 3a). The equivalent stress block parameters, α_2 was taken as 0.85 and γ was calculated using Equation (15) as given in ACI 318-11 [40].

$$\gamma = 0.85 - 0.005 f_{co}, \quad 0.65 \leq \gamma \leq 0.85 \quad (15)$$

The effective circular concrete area A_c of the equivalent stress block of width b_o , diameter D_o and angle subtended at the center of circle α was calculated according to Equations (16) - (18):

$$b_o = 2 \sqrt{\frac{D_o^2}{4} - \left(\frac{D_o}{2} - \gamma d_N \right)^2} \quad (16)$$

$$\alpha = 4 \tan^{-1} \left(\frac{2 \gamma d_N}{b_o} \right) \quad (17)$$

$$A_c = \frac{D_o^2}{8} (\alpha - \sin \alpha) \quad (18)$$

259 The resultant concrete compressive force (C_c) acting at the centroid of compression segment was
 260 calculated using Equation (19) and the point of application of C_c at a distance y' from the center of
 261 circular section was calculated using Equation (20).

$$C_c = \alpha_2 f'_{cc} A_c \quad (19)$$

$$y' = \frac{D^3}{A_c} \left(\frac{\sin(\alpha^3/2)}{12} \right) \quad (20)$$

262 where f'_{cc} is equivalent to f_{co} in Group REF and f'_{cc} is the FRP tube confined concrete strength in
 263 Groups CT and CTCR.

264 The resultant force in FRP tube in Groups CT and CTCR was calculated based on the equivalent stress
 265 and strain distribution. The contribution of FRP tube in resisting load under compression was ignored as
 266 FRPs are weaker in compression than in tension. The net force in FRP tube is equivalent to the tensile
 267 force in FRP tube (T_{tube}) and was calculated using Equation (21):

$$T_{tube} = \frac{1}{2} \epsilon_{bot} E_{FRP} t_{FRP} (D - d_N) \quad (21)$$

268 where ϵ_{bot} is the strain in FRP tube under tension and was calculated as a function of the FRP confined
 269 concrete strain (ϵ_{cu}) as given in Equation (22):

$$\epsilon_{bot} = \epsilon_{cu} \frac{d_N - (D + 2t_{FRP})}{d_N} \quad (22)$$

270 where d_N is the depth of neutral axis of section. The moment produced by the resultant concrete
 271 compressive force (C_c) about the centroid of the circular section in Group REF was calculated using
 272 Equation (23). The moment produced by the resultant concrete compressive force (C_c) and tensile force
 273 in FRP tube (T_{tube}) about the centroid of the circular section in Groups CT and CTCR was calculated
 274 using Equation (24):

$$M = C_c y' \quad (23)$$

$$M = C_c y' + T_{tube} \left(\frac{D_o}{6} + \frac{d_N}{3} \right) \quad (24)$$

275 The reinforcing bars in Specimens REF and CTCR were placed in four layers at distance d_i (d_1, d_2, d_3
 276 and d_4) from the extreme compressive fiber (Fig. 3b). The strain in each steel bar was calculated using
 277 Equation (25) and the strain in each CFRP bar was calculated using Equation (26).

$$\epsilon_s = 0.003 \frac{d_N - d_i}{d_N} \quad (25)$$

$$\epsilon_{CFRP} = \epsilon_{cu} \frac{d_N - d_i}{d_N} \quad (26)$$

278 The stress in steel bar (f_s) was calculated using Equation (27) and the stress at each CFRP bar (f_{CFRP})
 279 was calculated using Equation (28).

$$f_s = E_s \epsilon_s \leq f_y \quad (27)$$

$$f_{CFRP} = E_{bar} \epsilon_{CFRP} \leq f_{fu} \quad (28)$$

280 The force in steel bar (F_s) was calculated using Equation (29) and the force in the CFRP bar (F_{CFRP})
 281 was calculated using Equation (30).

$$F_s = f_s A_s \quad (29)$$

$$F_{CFRP} = f_{CFRP} A_{CFRP} \quad (30)$$

282 The moment produced by steel bars (M_s) about the centroid of the circular REF cross-section was
 283 calculated using Equation (31), whereas the moment produced by CFRP bars (M_{CFRP}) about the
 284 centroid of the circular CFRP-CFFT cross-section was calculated using Equation (32).

$$M_s = \sum F_s \left(\frac{D_o}{2} - d_i \right) \quad (31)$$

$$M_{CFRP} = \sum F_{CFRP} \left(\frac{D_o}{2} - d_i \right) \quad (32)$$

285

4.2. Comparison of Analytical and Experimental Axial Flexural Interactions

The analytical axial flexural ($P-M$) interactions of Group REF were developed using a similar approach adopted in ACI 318M-11 [40]. The analytical $P-M$ interactions of Groups CT and CTCR were developed using FRP confined concrete design codes i.e. ACI 440.2R-08 [41] and *fib* Bulletin 14 [42], and Samaan *et al.* [43] model. The $P-M$ interactions of Groups CT and CTCR are compared with the experimental $P-M$ interactions to validate the developed analytical model.

For Group REF, the analytical $P-M$ interaction underestimated the experimental $P-M$ interaction at concentric and eccentric axial loads and four-point load (Fig. 4). For Specimens REF-0, REF-25 and REF-50, analytical axial loads were 93.1%, 90.5% and 87.4%, respectively, of the experimental axial loads. For Specimens REF-25, REF-50 and REF-B, analytical bending moments were 84.5%, 85.4% and 89.3%, respectively, of the experimental bending moments. The analytical results showed that the specimens in Group REF can be modeled as unconfined concrete specimen by ignoring the confinement provided by the steel helix.

The analytical $P-M$ interaction of Group CT constructed with ACI 440.2R-08 [41] underestimated the experimental $P-M$ interaction at concentric and eccentric axial loads and four-point load (Fig. 5). Analytical axial loads of Specimens CT-0, CT-25 and CT-50 computed using ACI 440.2R-08 [41] were 76.2%, 90.3% and 75.5%, respectively, of the experimental axial loads. Analytical bending moments of Specimens CT-25, CT-50 and CT-B computed using ACI 440.2R-08 [41] were 84.6%, 76.0% and 60%, respectively of the experimental bending moments. The analytical $P-M$ interaction of Group CT constructed with *fib* Bulletin 14 [42] matched well with the corresponding experimental $P-M$ interaction (Fig. 5). Analytical axial loads of Specimens CT-0, CT-25 and CT-50 computed using *fib* Bulletin 14 [42] were 90.6%, 105.2% and 88.4%, respectively of the experimental axial loads. Analytical bending moments of Specimens CT-25, CT-50 and CT-B computed using *fib* Bulletin 14 [42] were 98.1%, 88.8% and 62.7%, respectively, of the experimental bending moments. The analytical $P-M$ interaction of Group CT constructed with Samaan *et al.* [43] model also matched well with the experimental $P-M$ interaction (Fig. 5). The analytical axial loads of Specimens CT-0, CT-25 and CT-50 computed using Samaan *et al.* [43] model were 81.5%, 95.7% and 74.6%, respectively, of the experimental axial loads.

The analytical bending moments of Specimens CT-25, CT-50 and CT-B computed using Samaan *et al.* [43] model were 96.7%, 79.9% and 84.5%, respectively, of the experimental bending moments. For Group CT, the analytical $P-M$ interaction constructed using *fib* Bulletin 14 [42] exhibited the best match with the corresponding experimental $P-M$ interaction. This is because *fib* Bulletin 14 [42] predicted the larger value of confined concrete strength ($f'_{cc} = 42.13$ MPa) than ACI 440.2R-08 [41] ($f'_{cc} = 36.26$ MPa) and Samaan *et al.* [43] model ($f'_{cc} = 40.10$ MPa) of CFRP tube confined concrete specimens. The larger the f'_{cc} , the larger is the compressive force in the confined concrete (C_c) (Equation (19) and corresponding bending moment (Equation (24)). Hence, larger is the axial load and bending moment in the specimens of Group CT.

The analytical $P-M$ interaction of Group CTCR constructed using ACI 440.2R-08 [41] underestimated the experimental $P-M$ interaction at concentric axial load. The analytical axial load of Specimen CTCR-0 computed using ACI 440.2R-08 [41] was 88.4% of the experimental axial loads. The analytical $P-M$ interaction of Group CTCR constructed using ACI 440.2R-08 [41] significantly underestimated the experimental $P-M$ interaction at 25 mm and 50 mm eccentric axial loads (Fig. 6). The analytical axial loads of Specimens CTCR-25 and CTCR-50 computed using ACI 440.2R-08 [41] were 73.2% and 100.2%, respectively of the experimental axial loads. The analytical bending moments of Specimens CTCR-25 and CTCR-50 computed using ACI 440.2R-08 [41] were 53.6% and 76.7%, respectively, of the experimental bending moments. The ACI 440.2R-08 [41] underestimates the ultimate confined concrete strain (ϵ_{cu}) and hence underestimated the bending moment of tested specimens. The analytical $P-M$ interaction of Group CTCR constructed using ACI 440.2R-08 [41] overestimated the experimental $P-M$ at four-point load as CFRP bars did not exhibit an adequate bond with the surrounding concrete and slipped under four-point load.

The analytical $P-M$ interaction of Group CTCR constructed using *fib* Bulletin 14 [42] matched well with the experimental $P-M$ interaction at concentric axial load. However, the analytical $P-M$ interaction of Group CTCR constructed using *fib* Bulletin 14 [42] underestimated the experimental $P-M$ interaction at 25 mm and 50 mm eccentric axial loads (Fig. 6). The analytical axial loads of Specimens CTCR-25 and CTCR-50 computed using *fib* Bulletin 14 [42] were 82.8% and 111.1%, respectively, of

the experimental axial loads. The analytical bending moments of Specimens CTCR-25 and CTCR-50 computed using *fib* Bulletin 14 [42] were 60.5% and 85%, respectively, of the experimental bending moments. The *fib* Bulletin 14 [42] also underestimates the ultimate confined concrete strain (ϵ_{cu}) and hence underestimated the bending moment of the tested specimens. The analytical $P-M$ interaction of Group CTCR constructed using *fib* Bulletin 14 [42] overestimated the experimental $P-M$ at four-point load as CFRP were slipped under four-point load.

The analytical $P-M$ interaction of Group CTCR constructed using Samaan *et al.* [42] model matched well with the experimental $P-M$ interaction at concentric and 25 mm eccentric axial loads. The analytical axial loads of Specimens CTCR-0 and CTCR-25 computed using Samaan *et al.* [43] model were 96.3% and 111.2%, respectively, of the experimental axial loads. Analytical bending moment of Specimen CTCR-25 computed using Samaan *et al.* [43] model were 87.2% of the experimental bending moment. The analytical $P-M$ interaction of Group CTCR constructed using Samaan *et al.* [43] model overestimated the experimental $P-M$ interaction at 50 mm eccentric axial load and four-point load (Fig. 6). The analytical axial loads of Specimens CTCR-50 computed using Samaan *et al.* [43] model were 141.4% of the experimental axial load. The analytical bending moments of Specimens CTCR-50 and CTCR-B computed using Samaan *et al.* [43] model were 108.5% and 120%, respectively, of the experimental bending moments. For Group CTCR, the analytical $P-M$ interaction constructed with Samaan *et al.* [43] model exhibited the best match with the corresponding experimental $P-M$ interaction. This is due to the fact that Samaan *et al.* [43] model predicted the larger value of ultimate confined concrete strain ($\epsilon_{cu} = 0.010$) than ACI 440.2R-08 [41] ($\epsilon_{cu} = 0.0056$) and *fib* Bulletin 14 [42] ($\epsilon_{cu} = 0.0054$) of CFRP tube confined concrete specimens. It is noted that the strain in the CFRP bars was calculated based on the strain compatibility between the confined concrete and CFRP bars. The larger the ϵ_{cu} , the larger are the strains and corresponding stresses in CFRP bars resulting in larger forces and bending moments in CFRP bars. Hence, larger the ϵ_{cu} , larger are axial loads and bending moments in the specimens of Group CTCR.

It can be summarized that ACI 440.2R-08 [41] underestimated the f'_{cc} and ϵ_{cu} of the CFRP tube confined concrete specimens and *fib* Bulletin 14 [42] underestimated the ϵ_{cu} of the CFRP tube confined concrete specimens. The f'_{cc} and ϵ_{cu} of the CFRP tube confined concrete specimens predicted with Samaan *et al.* [43] model were close to the experimental values. The analytical $P-M$ interactions constructed using Samaan *et al.* [43] model matched well with the experimental $P-M$ interactions.

5. Parametric Study

A parametric study was designed to investigate the effects of actual confinement ratio ($f_{l,a}/f_{co}$) and orientation of fibers (θ) of CFRP tube on axial flexural ($P-M$) interactions of CFRP tube confined concrete specimens (Group CT). Also, the effect of longitudinal CFRP bar reinforcement ratio on $P-M$ interactions of CFRP bar reinforced CFRP tube confined concrete specimens (CTCR group) was investigated. The parametric study considered a CFRP tube of 204 mm outer diameter and 812 mm height filled with 37 MPa concrete. For Group CT with outer fibers oriented at 60° to the longitudinal direction, four actual confinement ratios ($f_{l,a}/f_{co} = 0.10, 0.15, 0.20$ and 0.25) were considered. The selected actual confinement ratios are greater than the limiting actual confinement ratio ($f_{l,a}/f_{co} \geq 0.073$) which is the minimum level of confinement required to ensure an ascending second linear curve in the axial stress-strain performance of FRP confined concrete specimens [41]. Three orientations of fibers (θ) with reference to the axial direction (laminates) (i.e., 30°, 45° and 60°) were selected with inner layer of fibers oriented along the circumferential direction. A 60° laminate is selected to provide high level of confinement and, consequently, high axial load capacity under eccentric loading. A 30° laminate is selected to provide a low level of confinement and, consequently, high flexural load capacity under eccentric loading. Four longitudinal reinforcement ratios of CFRP bar in Group CTCR (1.32%, 2.35%, 3.28% and 5.26%) with actual confinement ratio of 0.10 and outer layer of fibers oriented at 60° to the longitudinal direction were selected. Group CTCR reinforced with CFRP bars of nominal diameter 9.5 mm, 12.7 mm, 15 mm and 19 mm, respectively, resulted in longitudinal reinforcement ratios of 1.32%, 2.35%, 3.28% and 5.26%.

To construct P – M interactions for four $f_{l,a}/f_{co}$ and three θ in CFRP tube of Group CT, and four longitudinal reinforcement ratios of CFRP bar in Group CTCR, the equivalent rectangular stress block method was selected to analyze the circular cross-section and Samaan *et al.* [43] model was selected. Normalized P^* – M^* interactions were constructed using normalized axial load (P^*) (Equation (33)) and normalized bending moment (M^*) (Equation (34)).

$$P^* = \frac{P}{f_{co} A_g} \quad (33)$$

$$M^* = \frac{M}{f_{co} A_g D} \quad (34)$$

The normalized P^* – M^* interactions of Group CT for four $f_{l,a}/f_{co}$ (0.10, 0.15, 0.20 and 0.25) are presented in Fig. 7. Increasing $f_{l,a}/f_{co}$ resulted in significant increase in axial loads at concentric axial load, and increased axial loads and lateral deformations and corresponding bending moment at 25 mm eccentric axial loads. However, increasing $f_{l,a}/f_{co}$ resulted in smaller increase in loads and bending moments at 50 mm eccentric axial loads and four-point load than concentric and 25 mm eccentric axial loads. This was attributed to the fact that increase in $f_{l,a}/f_{co}$ resulted in increase in confinement provided by the CFRP tube as two-thirds of the fibers were oriented along the circumferential direction, which were effective in confining the concrete under axial loads. However, the CFRP tube confinement was reduced under increasing applied axial load eccentricity. Two-thirds of the fibers oriented along the circumferential direction were not much effective and only one-third of the fibers oriented along the longitudinal direction were effective in resisting the load under four-point load.

The normalized P^* – M^* interactions of Group CT with fibers oriented at 30°, 45° and 60° along the longitudinal direction are presented in Fig. 8. Fig. 8 demonstrates that effect of fiber orientation was more pronounced at concentric and eccentric axial loads than the four-point load. This was because the orientation of fibers closer to the circumferential direction was effective in confining the concrete under axial loads.

The normalized P^*-M^* interactions of Group CTCR for four longitudinal reinforcement ratios (1.32%, 2.35%, 3.67% and 5.29%) are presented in Fig. 9. The increase in longitudinal reinforcement ratio resulted in increase in loads and lateral deformations and corresponding bending moment under concentric and eccentric axial loads and four-point load. This was attributed to the fact that CFRP bars were effective in resisting loads under concentric and eccentric axial loads and four-point load.

6. Conclusions

In this study the experimental and analytical axial-flexural ($P-M$) interactions of steel RC (Group REF), CFRP-CFFT (Group CT) and CFRP bar reinforced CFRP-CFFT (Group CTCR) were presented. A parametric study was conducted to investigate the effects of actual confinement ratio and orientation of fibers in CFRP tube and CFRP bar reinforcement (longitudinal reinforcement) ratio on $P-M$ interactions. Based on the experimental and analytical results, the following conclusions can be drawn:

The experimental $P-M$ interactions of Group CTCR were larger than the interactions of Groups REF and CT, as Group CTCR carried higher axial loads and higher bending moments than Groups REF and CT. The experimental $P-M$ interactions showed higher reduction in the effectiveness of CFRP reinforcement (tube and bar) than steel reinforcement with increase in the applied axial load eccentricity, as FRP reinforcement has lower modulus of elasticity than steel reinforcement.

The analytical $P-M$ interactions of Groups CT and CTCR constructed with ACI 440.2R-08 [40] significantly underestimated the experimental $P-M$ interactions. The analytical $P-M$ interactions of Group CT constructed with *fib* Bulletin 14 [41] matched well with the experimental $P-M$ interactions whereas the analytical $P-M$ interactions of Group CTCR were significantly underestimated. The analytical $P-M$ interactions of Group CT and CTCR constructed with Samaan *et al.* [42] matched well with the experimental $P-M$ interactions. Furthermore, the stress block method can be used to accurately compute the axial loads and bending moments of Groups CT, CTCR and REF.

The parametric study showed that the actual confinement ratio and orientation of fibers of CFRP tube have a profound effect on concentric and eccentric axial loads and negligible effect on four-point load, as fibers in specimens are oriented in the circumferential direction and are more effective in confining the

concrete under axial loads rather than reinforcing the specimens in the longitudinal direction. The parametric study also showed that the longitudinal reinforcement ratio has a significant effect on concentric and eccentric axial loads and four-point load.

Based on the analytical and experimental results, it is recommended that unreinforced CFRP-CFFT can efficiently serve as an alternative of steel bar RC columns only under concentric axial load. However, CFRP bar reinforced CFRP-CFFT can serve as an alternative of steel bar RC columns under concentric and eccentric axial loads in regions where corrosion of steel bar is a main concern.

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564 **Table 1.** Test matrix

Specimen ID	Outer diameter (mm)	Height (mm)	Transverse reinforcement	Longitudinal reinforcement	Applied load eccentricity
REF-0	205	800	Steel helix	Steel bar	0
REF-25	205	800	Steel helix	Steel bar	25
REF-50	205	800	Steel helix	Steel bar	50
REF-B	205	800	Steel helix	Steel bar	B
CT-0	204	812	CFRP tube	-	0
CT-25	204	812	CFRP tube	-	25
CT-50	204	812	CFRP tube	-	50
CT-B	204	812	CFRP tube	-	B
CTCR-0	204	812	CFRP tube	CFRP bar	0
CTCR-25	204	812	CFRP tube	CFRP bar	25
CTCR-50	204	812	CFRP tube	CFRP bar	50
CTCR-B	204	812	CFRP tube	CFRP bar	B

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Table 2. Test results of column specimens

Specimen ID	Peak axial load (kN)	Axial deformation at peak axial load (mm)	Lateral deformation at peak axial load δ (mm)	Bending moment M (kN.m)	Mid-height circumferential strain at peak axial load		Axial strain in bars at mid-height at peak axial load	
					Compression side	Tension side	Compression side	Tension side
REF-0	1529	2.4	0	0.0	0.0010	0.0001	0.0084	0.0156
REF-25	888	4.2	3.3	25.1	0.0005	-	0.0032	0.0002
REF-50	594	3.2	3.2	31.6	0.0075	0.0002	0.0003	0.0001
CT-0	1770	18.1	0	0.0	0.0143	0.0118	-	-
CT-25	755	3.5	4.7	22.4	0.0036	0.0007	-	-
CT-50	489	2.8	3.9	26.3	0.0014	0.0003	-	-
CTCR-0	2197	20.9	0	0.0	0.0133	0.0148	0.0023	0.0034
CTCR-25	1182	6.5	12.3	44.1	0.0072	0.0023	0.0029	0.0010
CTCR-50	666	4.3	19.2	46.1	0.0017	0.0005	0.0017	0.0005

568 **Table 3.** Test results of beam specimens

Specimen ID	Peak flexural load (kN)	Midspan deflection at peak flexural load (mm)	Bending moment M (kN.m)	Longitudinal strain at midspan of specimen at peak flexural load		Strain in bars at mid-height at peak flexural load	
				Compression side	Tension side	Compression side	Tension side
REF-B	347	37.3	27.0	0.0002	0.0013	0.0237	0.0148
CT-B	93	26.7	11.0	0.0072	0.0220	-	-
CTCR-B	223	28.2	26.3	0.0096	0.0283	0.0040	0.0013

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Fig. 1. Experimental $P - M$ curves of Groups REF, CT and CTCR

(i) Concentric axial load

(ii) 25 mm eccentric axial load

(iii) 50 mm eccentric axial load

(iv) Four-point load

Fig. 2. Experimental load deformation curves of Groups REF, CT and CTCR under (i) concentric axial load, (ii) 25 mm eccentric axial loads, (iii) 50 mm eccentric axial loads and (iv) four-point load

(i) Group CT

(ii) Group CTCR

Fig. 3. Stress and strain profiles for computing $P - M$ interactions of groups: (a) CT, and (b) CTCR

Fig. 4. Experimental and analytical $P - M$ interactions of Specimen CTCR

Fig. 5. Experimental and analytical $P - M$ interactions of Specimen CT

Fig. 6. Experimental and analytical $P - M$ interactions of Specimen CTCR

Fig. 7. Normalized $P^* - M^*$ interactions of Group CT for different actual confinement ratios

Fig. 8. Normalized $P^* - M^*$ interactions of Group CT for different orientation of fibers

Fig. 9. Normalized $P^* - M^*$ interactions of Group CTCR for different CFRP bar reinforcement ratios