Rectangular Concrete-Filled GFRP Tubes as Flexural and Axial Members

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ABSTRACT

In this paper, the feasibility of using concrete-filled rectangular filament-wound glass fibre reinforced polymer tubes (CFRFT) as structural members has been explored. Three beams and five short columns made from CFRFT were tested. In beam tests, two totally filled tubes and a tube partially filled with concrete and had a central hole for reducing deadweight were tested under four point bending. The effect of reinforcement ratio is examined by testing beams with GFRP tubes of two different sizes. The behaviour of CFRFT is compared to conventional concrete-filled rectangular steel tubes (CFRST) of similar reinforcement ratios. The short columns were tested under eccentricity ratios \((e/h)\) of 0, 0.092, 0.184 and 0.236, where \(h\) is the depth of the cross-section. The study showed that CFRFT is a practical and feasible system that offers similar flexural strength to CFRST of similar reinforcement ratio. An analytical model has been developed to predict the moment-curvature responses of beams and the complete interaction curves of beam-column members. The model accounts for different laminate structures of the flange and the web of the FRP tube through the Classical Lamination Theory (CLT). The gradual reduction of stiffness resulting from the progressive failure of different FRP layers oriented at various angles is accounted for through the Ultimate Laminate Failure (ULF) approach. The model adopts the cracked section analysis using layer-by-layer approach and accounts for both totally filled tubes and tubes with inner voids. The model has been utilized to evaluate the effects of laminate structure, hybrid laminates and tube thickness, and to optimize the inner voids of partially filled tubes. Comparisons of the behavior of CFRFT with conventional RC sections under different loading conditions showed that CFRFT could provide axial load-bending moment interaction curves comparable to those of RC sections of similar reinforcement index.

Keywords: FRP, rectangular tube, beam, column, concrete-filled, ultimate laminate failure (ULF), model
INTRODUCTION

The use of fibre-reinforced polymers (FRP) as structurally integrated stay-in-place formwork for concrete structures maximizes the advantages of both FRP and concrete, while simplifying the construction procedure, particularly when using closed tubular sections. In this case, circular or rectangular FRP tubes are filled with concrete and used as axial or flexural members. The tube provides lightweight permanent formwork and non-corrosive reinforcement simultaneously. Concrete-filled rectangular and circular steel tubes have been extensively studied in bending and under axial loads several researchers [1], [2], and [3].

A number of FRP-concrete hybrid systems have been developed over the years, including both open and closed FRP forms. While concrete-filled circular FRP tubes have been extensively studied in bending and under axial loads by a number of researchers [4], [5], [6] and [7], very limited studies have been conducted on rectangular, open or closed, FRP-concrete hybrid systems. Fardis and Khalili [8] proposed a rectangular FRP box section with one side open and filled with concrete for beam applications. Triantafillou and Meier [9] proposed a beam system consisting of rectangular glass-FRP (GFRP) box section with a carbon-FRP (CFRP) laminate bonded to the GFRP tension flange and a concrete layer cast on top of the compression GFRP flange. Hall and Mottram [10] presented a hybrid section combining GFRP pultruded sections having two T-up-stands and a continuous base, commercially available as floor panels, with concrete. Mirmiran et. al [11] tested five square concrete-filled GFRP tubes under various combinations of axial and transverse loads to develop a moment-thrust interaction diagram. In the compression control region, the columns proved as strong as equivalent conventional reinforced concrete columns with six percent reinforcement.

In this paper, rectangular filament-wound GFRP tubes, totally and partially filled with concrete, were tested. The tubes contain fibres oriented at various directions, including [0], [±45] and [90] degrees to provide flexural rigidity, shear resistance and possible confinement. The system is tested in bending as well as under concentric and eccentric axial compression. The study is focused on evaluating the effects of the inner void provided in beams to reduce the dead weight, the effect of reinforcement ratio in terms of the ratio of cross-sectional areas of GFRP and concrete, comparison with conventional concrete-filled steel tubes and examining the progressive failure behaviour of GFRP laminate resulting from the various orientations of the layers. An analytical model is also introduced to predict the behaviour of this system under different combinations of bending and axial loading, accounting for the progressive laminate failure effect. The model is used in a parametric study to investigate a wider range of parameters beyond the limitation of the experimental program and to compare the behaviour of CFRFT to that of conventional reinforced concrete (RC) sections.

EXPERIMENTAL PROGRAM

The experimental program includes three full-scale beam tests and five column tests conducted on concentrically and eccentrically axially loaded specimens. The specimens consisted of concrete-filled rectangular GFRP tubes (CFRFT) of two different sizes as shown in Fig. 1(a), fabricated using a combined filament-winding and hand lay-up technique, where bidirectional glass fibre sheets were inserted into the top and bottom flanges. The stacking sequences for the flanges and webs were shown in Fig. 1(a). The objective of the experimental program is to study the flexural behaviour of rectangular CFRFT specimens as well as to establish the axial load–bending moment interaction curve of CFRFT. The study is also aimed at optimizing the beam section by providing a central hole to reduce the self-weight of the beam.
• **Beam Specimens**

The detailed of the three beam specimens tested in this study are shown in Fig. 1 where cross-sections “a”, “b” and “c” denote totally and partially filled large tube and totally filled small tube respectively. Unlike beams B1 and B3, beam B2 consisted of the large GFRP tube, was partially filled with concrete through introduction of a void, offset towards the tension side of the member to minimize the self-weight of the member and to optimize the use of the concrete. A Styrofoam prism was used to create the void. To prevent crippling of the thin webs of beam B2 above the supports, concrete end blocks were cast by filling the void over a length of 375 mm from each end. In all the beams the concrete was cast vertically. The self-weight of beam B2 was only 44 percent of that of B1.

- **Column Specimens**

The five short columns tested in this program consisted of the small GFRP tubes, which were totally filled with concrete and were all 680 mm long as shown in Fig. 1(b). Columns C1 and C2 were both tested under concentric axial compression loads, however C1 was loaded through a pinned end connections, while C2 had fixed ends. Columns C3, C4 and C5 were tested under eccentric loads at eccentricities of 25 mm, 50 mm and 64 mm respectively. C1 and C2 are intended to compare the effect of end loading conditions. The test columns, along with beam B3 are used to provide various points on the interaction diagram of CFRFT, using the small GFRP tube. The column specimens were cut from the totally filled small tubes to the desired length, using a diamond blade saw.

• **Material Testing of GFRP Tube and concrete**

Both tension and compression tests were conducted on coupons cut from the flange and web of the large tubes, in the longitudinal direction, using a water-cooled diamond blade saw. For tension tests, ten 305x38 mm coupons were used. The tests were conducted according to ASTM D3039, where aluminium tabs were used to grip the coupon at both ends. For the compression tests, seven 29x38 mm coupons were used. Concrete strength was determined by testing concrete prisms, which were cut to a size of 152 mm on all sides from a portion of the large beam near to one of the supports. The equivalent concrete cylinder strength was found to be 52 MPa.

• **Test Setup and Instrumentation**

The beam specimens were tested using four-point bending over a span of 2100 mm and with a distance of 300mm between the loads under displacement control with a 2000 kN capacity hydraulic actuator at a rate of 0.50 mm/min. The specimens were instrumented with strain gauge type displacement transducers, electrical foil gauges, potentiometers and digital displacement transducers to record loads, deflections, strains and slips.
Columns were subjected to concentric and eccentric axial loads applied at various eccentricities using a 9000 kN capacity testing machine under stroke control at a rate of 0.165 mm/min. Except specimen C2 which was tested between fixed platens, columns C1, C3, C4 and C5 were tested using a setup with free rotation allowed at the ends. Strains in the axial and transverse direction were measured using electrical foil gauges and strain gauge type displacement transducers. In C1 specimen, one quarter of the perimeter was heavily instrumented with foil strain gages in the transverse direction at mid-height to evaluate the confinement effectiveness.

**EXPERIMENTAL RESULTS**

In the following sections, the beam and column test results as well as the results of the tension and compression coupon tests are presented. The different failure modes of the specimens are also presented and discussed.

- **GFRP Coupon Test Results**

The stress-strain curves of the tension coupons for both the flange and the web of the GFRP tube are shown in Fig. 2 (a). The tensile strength and stiffness of the flange are significantly higher than the web due to the presence of additional four longitudinal [0] GFRP layers. The tensile strengths are 316 MPa and 152 MPa for the flange and web respectively. The stress-strain curves reflect the progressive failure of the different layers, which was initiated by failure of the transverse [90] layers, followed by failure of the [45] layers, which corresponds to the ultimate strength of the web. However, in the flange, ultimate strength was reached when the longitudinal [0] layers ruptured later at a significantly higher load than that corresponding to failure of the [45] layers. Fig. 2(b) shows the stress-strain curves of the compression coupons for both the flange and web. The compressive strengths were 260 MPa and 187 MPa for the flange and web, respectively. The absence of longitudinal [0] layers in the webs, results in a rather non-linear behaviour as the contribution of the resin matrix, which is a nonlinear material, becomes more pronounced.

![Stress-strain behaviour of the of GFRP coupons](image_url)

(a) Behaviour under tension  
(b) Behaviour under compression

**Beam Test Results**

The load-mid span deflection behaviour of the three test beams is shown in Fig. 3. In general, the behaviour is slightly non-linear as a result of the relatively non-linear stress-strain curves of the GFRP tubes, shown in Fig. 2, as well as the non-linear characteristics of the concrete fill. The cracking load is relatively low compared to the ultimate load. The load-axial strain behaviour of the beams is shown in
Fig. 4, in terms of the top fibre compressive strains as well as the bottom fibre tensile strains. The normalized moment-curvature response of beams B1, B2 and B3 as well as additional concrete-filled rectangular steel tubes selected from the literature is compared in Fig. 5. In the following sections, the flexural behaviour of CFRFT is discussed in details.

**Effect of inner void**

The load-deflection behaviour as well as the load-axial strain behaviour of beams B1 (totally filled) and B2 (with void) is given in Fig. 4 and Fig. 5, respectively, and shows that both beams had identical flexural stiffness, which indicates that the “cracked” moment of inertia is similar for both beams. However, the cracking load of B1, point “a”, is higher than B2 due to the higher “uncracked” moment of inertia of B1. The flexural strength of B2, point “b”, is about 22 percent lower than B1, point “c”. This is attributed to the different failure modes of the two beams, where B1 failed in tension while B2 failed in compression. In B1, the GFRP bottom flange ruptured at an axial strain of 0.027, which is the ultimate tensile strain of the flange as confirmed by the coupon tests. The corresponding maximum axial compressive strain at the compression side of B1 at failure was 0.005, point “b”, which is only 25 percent of the ultimate compressive strain of the GFRP flange based on coupon test results. At point “b”, in Fig. 3 and Fig. 4, the GFRP compression flange of B1 showed signs of outward local buckling and separation from concrete. This was reflected by a drop in the load and a reversed direction of the axial compressive strain. However, the beam continued to carry additional load up to point “c”, where tension failure occurred. B2 also suffered outward local buckling of the GFRP compression flange, point “b”, at an axial strain of about 0.005, however, the load never recovered as the flange buckling was accompanied by cracking and inward buckling of the relatively thin concrete flange inside the tube, and marked as point “d” in figures 3 and 4. Although the flexural strength of B2 is 22 percent lower than that of B1, the dead weight of B1 is 56 percent lighter than B2, resulting in an over all strength-to-weight ratio for B2, 77 percent higher than B1. For B2 the amount of slip measured was negligible compared to B1. This is attributed to the initiation of inwards buckling and cracking of the thin concrete compression flange.

**Effect of reinforcement ratio**

The effect of reinforcement ratio is studied by comparing beams B1 and B3. Both beams, B1 and B3, had identical laminate structure but different cross-sectional size which resulted in two different reinforcement ratios of 9.6 and 15.3 percent, respectively. The load-deflection and load-axial strain behaviour of the two beams are shown in Figures 3 and 4. Similar to B1, B3 had flexural tension failure by rupture of the GFRP tube. Failure occurred at axial tensile strain of about 0.027, similar to B1.
However, the corresponding axial compression strain in the upper flange was 0.0088, which is 80 percent higher than B1. It should be noted that B3 did not suffer from local buckling of the GFRP compression flange as evident by the compressive strain behaviour, compared to B1. The width-to-thickness ($b/t$) ratio of the GFRP flanges of B1 and B3 were 30 and 19 respectively, therefore, B3 was less susceptible to local buckling.

Fig. 5 shows the normalized moment-curvature response of the mid-span section of B1 and B3. In order to exclusively present the effect of reinforcement ratio, the moment and the curvature are normalized with respect the cross-sectional dimension and depth $h$, respectively. In order to compare B1 and B3 with concrete-filled steel tubes in the same graph (as will be discussed in the next section), the moment is also normalized with respect to the concrete and tube strengths, $f'_c$ and $f_y$. $f_y$ denotes the yield strength of steel tubes or the tensile strength of the GFRP flange. Fig. 5 shows that for beams B1 and B3, increasing the reinforcement ratio by 55 percent has resulted in increasing the flexural strength by about 41 percent.

### Comparison between CFRFT and CFRST

The behaviour of concrete-filled rectangular FRP tubes (CFRFT) is compared to that of concrete-filled rectangular steel tubes (CFRST) of similar reinforcement ratios in Fig. 5. Lu and Kennedy [2] tested a 152 x 254 mm CFRST with 6.4 mm wall thickness in bending. The yield strength and concrete compressive strength were 431 MPa and 45 MPa respectively. The reinforcement ratio of this beam was 15 percent, which is similar to CFRFT-B3. Tomii and Sakino [1] have also tested a 100 x 100 mm CFRST with 2.9 mm wall thickness in bending. The yield strength and concrete compressive strength were 194 MPa and 24 MPa respectively. The reinforcement ratio of this beam is 9.8 percent, which is similar to CFRFT-B1. Fig. 5 shows that the behaviour of CFRST is stiffer than the CFRFT initially, due to the higher Young’s modulus of steel compared to GFRP. As the steel tube yields, CFRST shows plastic behaviour, whereas the CFRFT is almost linear to failure, which occurs by rupture of the FRP tube in tension.

- **Column Test Results**

The load-axial strain behaviour of the short columns is shown in Fig. 6. The axial compressive strains reported for C1 and C2 are the average value measured on both sides of each column. For C3, C4 and
C5 the figure shows the axial strains measured on both sides, referred to as “Max” and “Min”. The kern point of the columns was at 45 mm from center, therefore, columns C4 and C5 with 50 mm and 64 mm eccentricities developed tensile strains on the “Min” side. For C3, with only 25 mm eccentricity, the section was entirely under compression, except near the end, where significant plastic behaviour occurred. Although the axial strengths of C1 and C2 are comparable, the measured axial strains are quite different due to the different end loading conditions. In column C2, the testing machine applied the load uniformly over the entire cross-section through a stiff steel plate as shown in Fig. 1(b). The round corners of the tube were also subjected to transverse “hoop” tensile stresses, which introduced a high level of biaxial state of stresses, and significantly reduces the transverse strength of the tube and causes fracture along the corner. For column C1, the load was transferred to the steel plate through a steel pin as shown in Fig. 1(a). As the load was applied, the steel plate was gradually bent upwards and resulted in a non-uniform stress on the column cross-section, where the loading was mainly localized on the middle portion. The measured transverse strains along the periphery also indicates that though the round corners of the GFRP tube provides some local confinement to concrete near peak unconfined strength $f'_c$, however, at higher loads the long flat sides of the tube bend outwards rapidly as the concrete core expands indicating loss of confinement efficiency due to the loss of restraint. This loading pattern has resulted in diagonal failure of C1 after significant axial strain was developed in the column as the concrete wedge gradually slided over the diagonally fractured plane.

Columns C3, C4 and C5 were eccentrically loaded, also using the steel pin to allow for free curvature of the columns. However, a much stiffer steel plate was used to transfer the load from the pin to the column section in order to avoid any possible bending of the plate in those tests. All the three columns failed either by crushing of the tube or fracture of the tube on the on the compression side.

**ANALYTICAL MODEL FOR DIFFERENT LOADING CONDITIONS**

To predict the moment-curvature response and axial load-bending moment interaction diagram of CFRFT, a strain compatibility/equilibrium model has been developed. The model accounts for different laminate structures in the flange and web of the FRP tube and can be applied to tubes with hybrid fibres as well as tubes partially filled with concrete. A cracked section analysis using layer-by-layer approach is adopted to integrate the stresses over the cross-sectional areas of concrete and FRP.

The stresses in the FRP tube are based on the stress-strain curves obtained from coupon tests or predicted using Classical Lamination Theory [14] together with the progressive and Ultimate Laminate Failure approach (CLT-ULF), accounting for the different properties of the flange and web. Since First Ply Failure (FPF) highly underestimate the strength of the GFRP tubes as evident from Fig. 2, progressive and Ultimate Laminate Failure approach is adopted to determine the strength of the tube laminate. The predicted stress-strain responses of GFRP coupons based on CLT-ULF are shown shown in Fig. 2 together with the test results. The unconfined stress-strain model by Popovics [12] with extended strain softening was adopted for concrete in compression, while for concrete in tension the model proposed by Vecchio and Collins [13] has been adopted. The stresses at the centroid of each layer are assumed to be constant throughout its thickness.

**VERIFICATION OF THE MODEL**

- **Flexural Response**

The model is applied to the beam specimens to predict the moment-curvature. The predicted moment-curvature responses of beams B1, B2 and B3 using both the coupon test properties and the CLT-ULF predicted properties of the FRP tube are shown in Fig. 7. The figure shows that the model provides good prediction for the flexural responses. Fig. 7(a) also shows that the tension stiffening (T.S.) has marginal
effect on the moment-curvature response as evident in beam B1, and therefore, can be ignored. The predicted responses using the stress-strain relations of the GFRP tube derived using CLT-ULF, indicate that the flexural capacities of beams B1 and B3 were slightly underestimated. This is due to the fact that CLT underestimated the ultimate strength and strain of the web as evident from Fig. 2. Point ‘c’ on the predicted curves in Fig. 7 indicates initiation of failure of the web of the tubes.

Fig. 7 Moment-curvature response based on CLT-ULF and coupon test results

- Beam-Column Response

The predicted interaction curve \( I_o \) along with the test results of the beam-column specimens is shown in Fig. 8. The model, which is based on unconfined concrete strength, shows good agreement with test results of CFRFT. However, it slightly overestimates the pure axial strength. Considering the maximum compressive strength of concrete \( 0.85 f'c \) under axial loading as suggested by Hognestad [15], the predicted axial strength corresponding to \( 0.85 f'c \) is 1854 kN which is very close to the experimental results of 1896 kN. The predicted interaction curve \( I_o \) is largely dominated by compression failure except for a small tension failure region at very low axial loads.

COMPARISON WITH CONVENTIONAL RC SECTIONS

In order to have a better understanding of the behaviour of CFRFT, its behaviour is compared with conventional reinforced concrete (RC) sections for both beams and beam-column members. The criteria used to design the RC sections are also discussed.

- Flexural Behaviour

An attempt is made to compare the flexural behaviour of beam B1 to RC beams with the same cross-sectional dimensions and concrete strength as B1 and reinforced with Grade 60 steel rebar. To select the steel reinforcement ratio, four different criteria have been adopted:

1. Section RC1 designed according to ACI 318-99 [16] to provide same moment capacity as B1.
2. Section RC2 with the same reinforcement ratio as B1 (\( \rho_s = \rho_f \)).
3. Section RC3 with reinforcement of equal axial stiffness as B1 (\( E_s A_s = E_f A_f \)).
4. Section RC4 with the same reinforcement index as B1 (\( \omega_s = \omega_f \)).
Fig. 9 shows a comparison of the four RC beams with CFRFT beam B1 in terms of the moment-curvature responses. The reinforcement for the four RC beams are also shown in the same figure. Beams RC2, RC3 and RC4 are all doubly-symmetrically reinforced. Beam RC1 designed according to ACI 318-99 [16], has similar moment capacity to beam B1 with a total reinforcement ratio of 3.7 percent. The beam is essentially doubly reinforced as attempt to design RC1 as singly reinforced resulted in a lower moment capacity than B1 due to the Code limitation of maximum reinforcement ratio $0.75 \rho_b$. The response of RC1 is quite stiffer than beam B1 due to the higher Young’s modulus of steel compared to GFRP. The response of beam B1 is also compared with beams RC2 and RC3. It is clear that RC3 provides similar stiffness to that of B1, before yielding of steel, however, as the beam included only 0.8 percent reinforcement ratio it showed significantly lower strength than B1. Beam RC2 showed a significantly higher strength than B1, however, when compared with B1, the behaviour of RC2 and RC3 could be misleading as the ultimate strength $f_u$ of the GFRP tube is considerably different from the yield strength $f_y$ of the Grade 60 steel. Therefore, it would be more appropriate to compare B1 with RC4 of a similar reinforcement index, $\omega$, defined as $\omega = \rho_f f_u / \rho_s f_y = \rho_f f_y / f_c$, where $\rho_f$ and $\rho_s$ are the reinforcement ratios of CFRFT and RC beam, respectively. The reinforcement ratio $\rho_f$ of beam B1 was 9.6 percent. Using the similar reinforcement index concept, the equivalent reinforcement ratio in RC4 is 5.6 percent. The reinforcement is symmetrically distributed along the perimeter of RC4. The flexural strength of beam RC4 is found to be almost similar to beam B1. Therefore, the flexural strength of CFRFT is quite comparable to conventional RC beam with similar reinforcement index. Although beams RC1, RC2 and RC4 were all under-reinforced and failed in tension, they all showed very limited ductility due to the relatively high reinforcement ratio in comparison to RC3.

• Interaction Curves

The interaction curve of the CFRFT section is compared to those of RC sections of the same dimensions (152 x 254 mm) and reinforced with Grade 60 steel. Three RC sections were selected based on the following criteria:
1. Section designed to provide the same axial compressive strength of the CFRFT section, (Curve I₁ in Fig. 8). This criteria resulted in \( \omega_s = 0.8 \omega_f \).
2. Section designed to provide the same bending strength as the CFRFT section, (Curve I₂ in Fig. 8). This criteria resulted in \( \omega_s = 1.45 \omega_f \).
3. Section designed with the same reinforcement index as the CFRFT section \( (\omega_s = \omega_f) \), (Curve I₃ in Fig. 8).

All three sections are doubly-symmetrically reinforced. The three interaction curves I₁, I₂, and I₃ are compared with interaction curve I₀ of the CFRFT section in Fig. 8. Curve I₁ representing a RC section with very small flexural capacity under pure bending while curve I₂ represents a RC section with very high axial compressive strength. Unlike curve I₁, curve I₂ is entirely governed by compression failure. Curve I₃ shows moderate flexural capacity in bending but higher axial capacity than the CFRFT. The full interaction curve I₃ is also governed by compression failure.

**PARAMETRIC STUDY**

The analytical model has been used in a parametric study to examine a wider range of parameters including hybrid FRP laminate with glass and carbon fibres, laminates with different proportions of fibres in the axial and transverse directions, tubes with different wall thickness and beams with different inner void sizes.

- **Tube with Hybrid Laminate**

The behaviour of the CFRFT with hybrid laminate including carbon fibres with a tensile strength of 3500 MPa and Young’s modulus of 227 Gpa, as replacement for the E-glass longitudinal [0] layers in the flanges has been investigated for both the large and small beams B₁ and B₃, respectively. The corresponding moment-curvature responses of the hybrid beams are shown in Fig. 10, where the flexural strength was found to increase by 56 and 62 percent, respectively. Thus a small amount of carbon fibres placed longitudinally in the flanges could increase the flexural strength substantially without much increase of the overall cost.

![Fig. 10](image1)  
**Fig. 10** Moment-curvature response of hybrid CFRFT compared to all-GFRP CFRFT

![Fig. 11](image2)  
**Fig. 11** Effect of the thickness of concrete flange on moment capacity of CFRFT
• **Optimization of Inner Void in the Rectangular Section**

Fig. 11 shows that the moment capacity of the partially filled tube B2 increases as the flange thickness-to-depth ratio \((h_t/D)\) increases up to a ratio of 0.15, after which, almost no increase in flexural strength is observed. Thus extending the concrete flange below the neutral axis level is ineffective and the section can be optimized with relatively small \(h_t/D\) ratio. Voided sections provide a substantially reduced self-weight and increased efficiency in bending.

• **Effect of Laminate Structure and Wall Thickness on Flexural Behaviour**

A rectangular GFRP tube with inner dimensions of \(152\times254\) mm and \([0/90]_s\) symmetric cross-ply E-glass/epoxy laminate is used in the parametric study with five different laminate structures where the ratios of the fibres in the axial to transverse directions were 9:1, 3:1, 1:1, 1:3 and 1:9. For each laminate structure, five different wall thicknesses have been chosen, including 1, 2, 4, 8 and 16 mm which were equivalent to reinforcement ratios of 2.1, 4.2, 8.6, 17.5 and 36.3 percent, respectively. The CLT-ULF approach has been used to determine the constitutive properties of the laminates.

The variation of the flexural strength with the reinforcement ratio \(\rho\) is shown in Fig. 12 for the 50 MPa concrete and for different laminate structures. The curves are normalized with respect to the cross-sectional dimensions, concrete strength \(f_c'\), and the ultimate tensile strength \(f_u\) of the tubes. For a particular laminate structure, increasing the wall thickness results in increasing the flexural capacity and could lead to change in the failure mode from tension to compression. Also for a given wall thickness, the flexural capacity increases as the percentage of fibres in the axial direction \([0]\) increases. The rate of increase is much higher for thick tubes than for thin tube. Fig. 12 also presents the locus of the balanced reinforcement ratio, which reduces gradually with increasing the stiffness of the tube in the axial direction.

The effect of reinforcement ratio on moment capacity is compared in Fig. 13 for both CFRFT and conventional RC sections including singly and doubly reinforced sections. A CFRFT with \((1:1.5)\) laminate is used in this analysis which has a tensile strength \(f_u\) similar to that of the yield strength \(f_y\) of Grade 60 steel. For the singly reinforced beam, the flexural capacity increases with increasing \(\rho\) in the tension failure region with a rate relatively higher than the CFRFT beam. Beyond the balanced reinforcement ratio (in the compression failure zone) the rate of increase of flexural strength of singly
reinforced RC section is negligible compared to the CFRFT section, which continues to increase with almost at a constant rate. The doubly-symmetrically reinforced RC section shows very similar trend to CFRFT. Therefore, flexural CFRFT members can provide a reliable replacement of conventional doubly reinforced RC sections and can also eliminate the problem of reinforcement congestions of RC beams at higher reinforcement ratios, which can ease the fabrication process.

- **Effect of Wall Thickness and Laminate Structure on Interaction Curves**

A rectangular tube similar to that used in the parametric study in flexure, with three different laminate structures of 1:9, 1:1 and 9:1 are used for two different wall thicknesses, 2 and 16 mm. The corresponding interaction curves are shown in Fig. 14. For a given laminate structure, the flexural capacity is increased with increasing the wall thickness. Also for a given wall thickness, increasing the percentage of fibres in the axial direction results in significant increase in the flexural capacity for both thin and thick tubes. However, its effect on the axial strength is only pronounced in thick tubes.

![Interaction curves of CFRFTs with different wall thickness](image)

Fig. 14 Interaction curves of CFRFTs with different wall thickness

Fig. 14 also indicates that CFRFT with thin tubes, under small and intermediate eccentricities, have a similar curve for all range of laminate structures. However, at large eccentricities, when the curve is dominated by tension failure, the bending capacities vary significantly. In this case, the most efficient laminate is the one with maximum amount of fibres oriented in the axial direction such as the 9:1. In CFRFT with thick tubes, the most efficient design is governed by tubes with maximum amount of fibres oriented in the axial direction (9:1), for both small and large eccentricities.

**CONCLUSIONS**

Based on the experimental and analytical studies on concrete-filled rectangular FRP tubes (CFRFT), the following conclusions are drawn:

1. CFRFT provides reliable and feasible structural members for beam and column applications. The system is quite simple in construction as the FRP tube provides permanent formwork and is the sole reinforcement for concrete in the axial and transverse directions.

2. The laminate stress-strain behaviour of FRP tubes could be quite nonlinear. Laminates with fibres oriented at ±45 degrees show significant nonlinearity under tension and compression. Nonlinearity could also result from progressive failure of layers oriented at various directions.
3. The flexural strength of CFRFT is comparable to that of concrete-filled steel tubes (CFRST) of similar reinforcement ratio. Their load-deflection behaviour, however, is quite different. CFRST are initially stiffer due to the higher Young’s modulus of steel compared to GFRP. Once the steel yields, plastic behaviour is observed, whereas CFRFT do not show plasticity.

4. The proposed model using either the experimental coupon test results or the CLT-ULF results predicts well the behaviour of CFRFT. Coupon test results are, however, slightly more accurate.

5. Conventional RC sections of the same moment capacity as CFRFT have higher axial compressive strength, while RC sections with the same axial strength as CFRFT have substantially lower flexural strength.

6. RC sections with similar flexural strength, reinforcement ratio, or reinforcement index to CFRFT are significantly stiffer, but have very low ductility. RC sections with reinforcement ratio based on equivalent axial stiffness ($EA$) of reinforcement, are quite ductile but substantially lower in flexural capacity than CFRFT.

7. In CFRFT flexural members, the void size can be optimized for maximum strength-to-self weight ratio. Tube with hybrid laminate could enhance significantly the flexural strength and stiffness without much increase in overall cost.

8. Increasing the thickness for a given laminate or increasing the percentage of fibres in the axial direction of the laminate results in increasing the flexural strength and stiffness of CFRFTs. However, failure mode could change from tension to compression.

9. The balanced reinforcement ratio of CFRFT under pure bending depends on the laminate structure. It reduces gradually with increasing axial stiffness of the laminate of the tube.

10. Variation of flexural strength with reinforcement ratio in CFRFT is quite different from singly reinforced RC sections but very similar to that of doubly reinforced RC section.

11. Increasing the wall thickness or percentage of fibres in axial direction of the tube, results in large gain in flexural strength for both thick and thin tubes. However, in thin tubes, increasing the percentage of fibres in axial direction has insignificant effect on axial strength.

ACKNOWLEDGEMENT

The authors wish to acknowledge the financial support provided by the Network of Centres of Excellence on Intelligent Sensing for Innovative Structures (ISIS Canada), the Constructed Facilities Laboratory of North Carolina State University, Queen’s University and Composite Atlantic Ltd.

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Fig. 7 Moment-curvature response based on CLT-ULF and coupon test results
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