Concrete-Filled Steel Tubes Subjected to Axial Compression and Lateral Cyclic Loads

Amir Fam¹; Frank S. Qie²; and Sami Rizkalla³

Abstract: This paper presents an experimental work and analytical modeling for concrete-filled steel tubes (CFSTs) subjected to concentric axial compression and combined axial compression and lateral cyclic loading. The objective of the study is to evaluate the strength and ductility of CFST short columns and beam-column members under different bond and end loading conditions. Both bonded and unbonded specimens were tested, including application of the axial load to the composite steel-concrete section and to the concrete core only. Research findings indicate that the bond and end loading conditions did not affect the flexural strength of beam-column members significantly. On the other hand, the axial strengths of the unbonded short columns were slightly increased, compared to those of the bonded ones, while the stiffness of the unbonded specimens was slightly reduced. Test results were compared with the available design specifications, which were found to be conservative. The paper also presents an analytical model capable of predicting the flexural and axial load strength of CFST members. Experimental results were found to be in good agreement with the predicted values.


CE Database subject headings: Steel; Tubes; Concrete; Lateral loads; Cyclic loads; Axial compression; Beam columns.

Introduction

Concrete-filled steel tubes (CFSTs) are used in many structural applications including columns supporting platforms of offshore structures, roofs of storage tanks, bridge piers, piles, and columns in seismic zones (Kilpatrick and Rangan 1997a). Recently, a sixty meter span space-truss pedestrian bridge was constructed in Quebec, Canada, utilizing high-performance reactive powder concrete, confined and press in steel tubes, and used as diagonal members of the stressed bridge (Dallaire et al. 1998). Application of the CFST concept can lead to 60% total saving of steel in comparison to a structural steel system (Zhong 1998). Steel tubes were also used as permanent formwork and to provide well-distributed reinforcement, located at the most efficient position (Furlong 1967). Test results (Fam 2000) have shown that the concrete core delays the local buckling and forces the steel tube to buckle outwards rather than inwards, resulting in a higher flexural strength. Therefore, tubes with thinner walls could reach the yielding strength before local buckling occurs (Lu and Kennedy 1992). Under axial compression, the steel tube confines the concrete, therefore, improves both the axial load resistance and ductility of the CFST members. Fam and Rizkalla (2002) reported a 50% increase in the flexural strength by filling a hollow steel tube with concrete. This is typically achieved with insignificant increase in cost, and without any increase in size. Due to the large shear capacity of concrete-filled steel tubular members, they predominantly fail in flexure in a ductile manner (Tomii and Sakano 1979). Furlong (1967) reported that using expansive cement enhances the bond and provides chemically prestressed elements. The composite action and bond in CFST were also studied by Roeder et al. (1999). Furlong (1967) reported that if the steel tube is axially loaded, the confinement effect is delayed, until the expansion of concrete overcomes that of the tube. Different researchers concluded that confinement effectiveness is reduced by using rectangular or square tubes, by using high strength concrete, by increasing the slenderness of columns and for pure flexural members (Furlong 1967; Knowles and Park 1969; Kennedy and MacGregor 1984; Lu and Kennedy 1992; Kilpatrick and Rangan 1997a, and Schneider 1998). Several mathematical models have been developed to predict the strength of CFST columns, including Furlong (1968), Xiao (1989), and Zhong (1985). Design specifications, including CAN/CSA (1994) and AISC LRFD (1998), are very conservative due to the lack of consideration of the confinement effect in CFST.

Experimental Program

A total of ten specimens were tested, including five short CFST column specimens and five CFST beam-column specimens. The various parameters considered in this investigation were bond, end-loading conditions, and the level of axial compressive stress for the beam-column specimens. For the unbonded columns, the bond between the steel and concrete was prevented by a layer of asphalt applied to the inside surface of the steel tube. The two different loading conditions included axial load applied to the concrete core only and axial load applied to the composite concrete core and steel tube. Short columns specimens were tested.
under monotonic axial compression loading conditions. Beam-column specimens were tested under combined constant axial load and reversed lateral cyclic loading conditions.

**Material Properties**

The steel tubes used for the construction of the CFST specimens were cold formed and welded steel tubing. The outside diameter of the tubes $D_o$ was 152.4 mm and the thickness of the wall was 3.12 mm. The diameter and wall thickness of the tube resulted in a $(D_o/t)$ ratio of 49. This ratio satisfies the limitations specified by different specifications, including the American AISC LRFD (1998) $[D_o/t\leq 8(f/y)^{0.5}]$, the Canadian CAN/CSA-S16.1-94 (1994) $[D_o/t\leq (28,000/f_y)]$, and the Japanese AJI (Qe 1994) $[D_o/t\leq (25,520/f_y)]$, where $f_y$ is the yield strength and $E$ is Young’s modulus of the tube. Mechanical properties of the hollow tubes were obtained using three tension coupon tests and three short tubes for compression tests. The tests were conducted according to ASTM specifications A547-89 (1990). The yield strengths were 347 and 382 MPa in tension and compression, respectively. The modulus of elasticity were 189 and 198 GPa in tension and compression, respectively. The ultimate elongation in tension was 29% and the average Poisson’s ratio was 0.25.

Ready-mixed concrete was used to fill the steel tubes. A total of twenty standard size cylinders were cast and cured under the same conditions as the CFST columns. Based on the cylinder test results at the time of testing, the concrete compressive strengths were 55 and 60 MPa for the short CFST and the CFST beam-column specimens, respectively. The tensile strength and Poisson’s ratio were 4.7 MPa and 0.2, respectively.

**Short Column Tests**

A length-to-diameter ratio of 3 was selected for the CFST short columns in order to ensure short column behavior. A total of five short columns were tested in this study including three bonded and two unbonded specimens.

Table 1 provides details of the short column specimens C1 to C5. Out of the three bonded tubes, one was axially loaded on the composite section C1 and two were axially loaded on the concrete core only C2 and C3. For the two unbonded specimens C4 and C5 the load was applied to the concrete core only. All specimens were cast in a vertical position and vibrated using hand vibrator. After casting, the top ends of the specimens were covered by plastic sheets and steel plates during curing.

**Fig. 1. Test setup and instrumentation of short column specimens**

**Table 1. Details and Test Results of Short Column Specimens**

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Type</th>
<th>Bond</th>
<th>End loading condition</th>
<th>$P_y$ (kN)</th>
<th>$P_{max}$ (kN)</th>
<th>$P_R$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>Short columns</td>
<td>Bonded</td>
<td>Composite section</td>
<td>2,025</td>
<td>2,389</td>
<td>1,295</td>
</tr>
<tr>
<td>C2</td>
<td>$D_o=152$ mm</td>
<td>Bonded</td>
<td>Concrete section</td>
<td>2,260</td>
<td>2,456</td>
<td>1,667</td>
</tr>
<tr>
<td>C3</td>
<td>$L_o=457$ mm</td>
<td>Bonded</td>
<td>Concrete section</td>
<td>2,260</td>
<td>2,442</td>
<td>1,550</td>
</tr>
<tr>
<td>C4</td>
<td>Unbonded</td>
<td></td>
<td>Concrete section</td>
<td>2,450</td>
<td>2,505</td>
<td>1,600</td>
</tr>
<tr>
<td>C5</td>
<td>Unbonded</td>
<td></td>
<td>Concrete section</td>
<td>2,450</td>
<td>2,549</td>
<td>1,771</td>
</tr>
</tbody>
</table>

$\text{a}$: $P_y$ = yielding load.

$\text{b}$: $P_{max}$ = maximum load.

$\text{c}$: $P_R$ = residual load.

The beam-column specimens were tested to study the behavior under a constant axial compression load and a lateral cyclic load. A length-to-diameter ratio of 12 was selected for the beam-column specimens in this study, which is recommended as an upper limit to avoid possible instability failure (Wakabayashi and Matosi 1988). The five beam-column specimens tested included three bonded and two unbonded specimens. Table 2 provides details of the beam-column specimens. Two bonded specimens BC1 and BC4 were axially loaded in compression using applied loads of 500 and 600 kN, respectively, acting on the composite steel-concrete section. Two unbonded specimens BC1 and BC5 were also axially loaded by 500 and 600 kN, respectively, acting on the concrete core only. One bonded specimen BC2 was loaded by
The axial compression load of 500 kN, acting on the concrete core only. The applied constant axial loads were about 20 to 25% of the ultimate axial load resistance of the CFST short column specimens. In all beam-column specimens, lateral cyclic loading was increased until failure occurred.

### Table 2. Details and Test Results of Beam-Column Specimens

<table>
<thead>
<tr>
<th>Spec No.</th>
<th>Type</th>
<th>Bond</th>
<th>End loading condition</th>
<th>Axial load (kN)</th>
<th>$M_0$ (kNm)</th>
<th>$M_0$ (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BC1</td>
<td>Beam columns</td>
<td>Bonded</td>
<td>Composite section</td>
<td>500</td>
<td>44.1</td>
<td>54.0</td>
</tr>
<tr>
<td>BC2</td>
<td>$D = 152$ mm</td>
<td>Bonded</td>
<td>Concrete section</td>
<td>500</td>
<td>39.0</td>
<td>56.6</td>
</tr>
<tr>
<td>BC3</td>
<td>$L = 1829$ mm</td>
<td>Unbonded</td>
<td>Concrete section</td>
<td>500</td>
<td>44.0</td>
<td>52.0</td>
</tr>
<tr>
<td>BC4</td>
<td>Bonded</td>
<td>Bonded</td>
<td>Composite section</td>
<td>600</td>
<td>32.8</td>
<td>61.6</td>
</tr>
<tr>
<td>BC5</td>
<td>Unbonded</td>
<td>Bonded</td>
<td>Concrete section</td>
<td>600</td>
<td>43.9</td>
<td>59.9</td>
</tr>
</tbody>
</table>

$M_y =$ Yield moment.

$M_{max} =$ Maximum moment.

Experimental Results and Discussion

### Short Column Specimens

The measured axial load versus axial deformation responses based on the LVDT readings of all tested short CFST columns are shown in Fig. 5. In general, the initial behavior was linear elastic up to the peak load, after which the load was dropped approximately one third of the maximum axial strength. All specimens maintained the load with excellent ductility up to failure. The axial load versus the axial and hoop strains, based on the strain gauge readings at five stations along the height of specimen C1,
C2, and C4, are shown in Fig. 6. The hoop strains developed in the bonded steel tube are a direct result of the effect of Poisson’s ratio of the tube under axial loading as well as the confinement effect due to the expansion of the concrete core. In the case of unbonded tube, where only the concrete core was axially loaded, the hoop strains were mainly developed due to confinement. Table 1 provides the axial loads at the onset of yielding, the maximum axial load and the residual load at the maximum tested deformation for all the short CFST specimens. It should be noted that the bonded steel tubes were subjected to a biaxial state of stress including axial compression and hoop tension. In order to define the yielding load, Von Mises yield criteria, given in Eq. (1), have been adopted. The measured strains in both directions ($\varepsilon_x, \varepsilon_y$) were used to determine the biaxial stresses ($\sigma_x, \sigma_y$) at different loads, using Eqs. (2a) and (2b), and compared to the Von Mises yield surface until yielding is detected. $E$, $\nu$, and $f_y$ are Young’s modulus, Poisson’s ratio, and the yield strength of the steel, respectively:

$$\frac{\sigma_x}{f_y} + \frac{\sigma_y}{f_y} - \frac{\sigma_x \sigma_y}{f_y^2} = 1$$  \hspace{1cm} (1)

$$\sigma_x = \frac{E}{1-\nu^2} \varepsilon_x + \frac{E \nu}{1-\nu^2} \varepsilon_y$$  \hspace{1cm} (2a)

$$\sigma_y = \frac{E}{1-\nu^2} \varepsilon_y + \frac{E \nu}{1-\nu^2} \varepsilon_x$$  \hspace{1cm} (2b)

The load drop in specimens C1, C2, and C3 was associated with formation of a major diagonal crack inside the concrete core as shown in Fig. 7. Under the confinement effect of the steel tube, the cracked surfaces slid against each other as the applied axial loading was increased. The friction between the surfaces of the cracked concrete is believed to be the main mechanism that main-
Fig. 7. Failure modes of short CFST specimens.

Failure occurred at the ends, where the concrete core failed in combined diagonal and vertical cracks as shown in Fig. 7. In all the five test specimens, failure was initiated by yielding of the steel tube, followed by the internal diagonal (or diagonal and vertical) cracking. In the C1 specimen, a slight increase in axial strength occurred after the minimum load resistance was reached at large axial deformation of 18 mm due to the strain hardening of the deformed steel. In the C2 and C3 specimens, the minimum sustained strength was relatively stable. The two unbonded specimens C4 and C5 had an initial linear stiffness lower than that of the bonded specimens and achieved slightly higher axial load resistance. For these specimens, the minimum load capacities became very stable and were maintained almost constant up to termination of the test after large deformation.

Effect of bond and end loading conditions: The effect of the end loading condition can be examined by comparing the behavior of bonded specimen C1 to that of bonded specimens C2 or C3. The behavior for all three specimens was identical within the elastic range, as shown in Fig. 5, since the concrete was fully bonded to the steel tube in the three specimens. The yielding and maximum loads of specimens C2 and C3, where the load was applied to the concrete core only, were 12 and 2.5% higher, respectively, than specimen C1, where the load was applied to the composite section. After initiation of diagonal cracking in concrete, C2 and C3 sustained more stable, and higher residual strength than C1. This is attributed to the higher confinement level of the concrete, induced by the steel tube in C2 and C3. In C1, the tube was axially loaded during the entire loading history, and therefore, the level of confinement was lower than C2 and C3 due to the Poisson’s ratio effect of the steel tube, which results in an outward radial displacement and less contact with concrete. In C2 and C3, the steel tube was originally debonded from the concrete. Consequently, it was released from sharing the axial compression, and therefore provided higher radial confinement pressure imposed on the diagonally cracked concrete. This has provided higher friction between the cracked concrete surfaces...
and resulted in higher residual axial strength. Schematic of this mechanism is illustrated at the top right corner of Fig. 5. Fig. 6 shows that, at the maximum measured load, both the axial and hoop strains of specimen C1 were about 62% higher than C2 due to the loss of the composite action after debonding occurred in C2. Also the hoop strains of C1 were significantly higher than in C2 due to the effect of Poisson’s ratio of the steel tube throughout the loading history of C1.

The effect of the bond can be examined by comparing C4 and C5 to C2 and C3. Fig. 5 shows that the initial stiffness of C4 and C5 is about 30% lower than those of the other columns. The reduction reflects the noncomposite behavior of the unbonded specimens under the axial loads. The behavior suggests that the confined concrete core mainly provided the axial stiffness, as the axial load transferred to the steel tube was very small. This is evident by the very small axial strain measured for C4, and shown in Fig. 6. The yielding axial loads of C4 and C5 were about 8% higher than those of C2 and C3 and about 21% higher than that of C1. The effect of bond on the yielding strength is not as large as observed by other researchers (Sato et al. 1987), who reported 30% increase of the yield strength for the unbonded columns as compared to bonded ones, due to the difference of the steel ratio of the CFST specimens. The steel ratio used in their test specimens was 17.4%, which is about twice the ratio used in this experimental program, 8.7%. The maximum axial loads of C4 and C5 were slightly higher than that of C1, however, the residual strength of C4 and C5 were significantly higher than that of C1, due to the higher level of confinement.

Beam-Column Specimens

The applied lateral load versus lateral deflection hysteresis of all the tested beam-column specimens are shown in Fig. 8. Fig. 8(f) shows the envelopes of the hysteresis of all beam-column specimens. The cyclic loading behavior of the specimens is relatively similar. All specimens showed linear behavior within the elastic range and maintained constant flexural stiffness without strength deterioration. The measured 2 mm residual deflection for all the specimens in the elastic range is attributed to loose fitting at the end hinges of the test frame. The lateral-load–deflection hysteresis loops were very stable without any strength deterioration up to an average ductility level of 3. Table 2 provides the measured yielding and maximum moments for all the beam-column specimens, which were determined from the lateral loads, accounting for the $P-\Delta$ effect of the axial loads.

The maximum moment and local buckling in the steel tube occurred almost simultaneously for specimen BC1, at lateral deflection level of 3.72$\Delta_y$. The maximum measured moment for specimen BC2 occurred prior to the local buckling of the tube at lateral deflection level between 3$\Delta_y$ and 4.5$\Delta_y$. The maximum moment for specimen BC3 occurred at deflection of 2.43$\Delta_y$, while local buckling occurred at 3.72$\Delta_y$. The local buckling of the tube for specimen BC4 occurred at 5.07$\Delta_y$, after reaching the maximum flexural strength. The maximum moment for specimen BC5 occurred at deflection of 2.5$\Delta_y$, before local buckling, which occurred at 3.75$\Delta_y$.

Fig. 9 shows the axial strain distribution along the length of beam-column specimen BC1 in the tension and compression sides, at different lateral loading levels. At low levels of the applied lateral loads, both sides of the specimen were in compression, due to the presence of the applied 500 kN concentric axial compression load. As the lateral load increases, the compressive strains increase on one side and decrease on the opposite side due to bending. At higher levels of lateral loads, tensile stresses develop on one side and both the tension and compression sides yield at the critical section at mid height. The length of the plastic hinge in all beam-column specimens was estimated by the average yield length of the steel tube, measured at maximum moment location on both the tension and compression sides. The plastic hinge lengths of specimens BC1, BC2, and BC5, which represent different bond and end loading conditions, were 114, 223, and 146 mm, respectively. These lengths correspond to 6.8, 13.4, and 8.8% of the length of the specimens and to 75, 146, and 96% of the diameter of the specimens, respectively.

At the beginning of the last cycle of testing of the beam-column specimens, small surface cracks appeared on the surface of the steel tube, caused by low cycle fatigue, as a consequence of successive buckling and straightening during the cyclic loading. These small cracks were further extended and finally led to a major transverse fracture of the steel tube. The tests were stopped as the specimens exhibited a significant loss of flexural resistance. Fig. 10 shows a typical beam-column specimen after failure.

Effect of bond and end loading conditions. The moment-curvature envelopes for all the tested beam-column specimens are shown in Fig. 11. The figure shows almost identical flexural stiffness to bending.
ness for all tested specimens, regardless of the different bond and loading conditions. The yielding moment of BC2 (bonded and loaded on concrete only) was about 11% lower than that of BC1 (bonded and loaded on the composite section). The maximum moment of BC2 was 4.8% higher than that of BC1. In BC2, yielding of the steel tube initiated on the tension side, while in BC1 yielding initiated on the compression side. Fig. 11 also indicates that the behavior of BC4 (bonded and loaded on the composite section) and BC5 (unbonded and loaded on the concrete section only) is very similar. However, the yielding moment of BC5 was 34% higher than that of BC4. The total number of the lateral loading cycles applied to BC3 was 10% lower than that of BC4, which indicates a lower level of energy absorption. The maximum ductility levels of BC4 and BC5 were 8.45 and 6.13, respectively.

Load Resistance Mechanisms and Analytical Modeling

Bonded CFST Short Columns

Based on test results, the bonded specimens, loaded on either the composite section (C1) or the concrete core (C2), exhibited virtually identical axial behavior in the elastic range. At this stage, both the concrete core and the steel tube shared the axial load. The axial compressive stresses in the tube of C1 is a result of direct loading of the composite section, while in C2, axial stresses are developed in the tube through the bond mechanism between the steel tube and the concrete. Due to the gradual development of internal microcracks in the concrete core, the rate of lateral expansion of the concrete increases and exceeds its initial Poisson’s ratio. Once the dilation of concrete exceeds Poisson’s ratio of the steel, confinement pressure is developed at the interface between the tube and the concrete and hoop tensile stresses develop in the tube. As the column was reaching the maximum load, the steel tube yielded and the concrete core failed in shear. The friction within the diagonally fractured concrete core, which is confined by the steel tube, became the major load resistance mechanism.

Literature review revealed several mathematical models developed by researchers and design specifications. Most of the models are proposed to predict the axial strength and stiffness of the bonded CFST short columns. Furlong (1968) considered the CFST column as a composite column and assumed that the concrete core and steel tube acted separately to resist the axial load. No confinement effect was considered in this model. Empirical formulas were provided to calculate the axial stiffness and strength of CFST. Xiao (1989) developed a model, which is based on Von Mises yielding criterion in steel tubes and an empirical concrete strength formula. Zhong (1985) introduced a model based on the Mohr’s circle strength theory for both the concrete core and steel tube and provided equations to calculate the axial strength and stiffness.

In addition to these models, design specifications including Canadian Institute of Steel Construction (CANC/CSA-S16.1-94, 1994), American Institute of Steel Construction (AISC LRFD 1998), and the Architectural Institute of Japan (AIJ) (1985) have also been used to predict the strength of short CFST columns. In order to compare the specified strength in different models, all the resistance factors in different design specifications are assumed to be unity. The predicted values are compared to the measured axial strength of the bonded CFST short column C1, loaded on the composite section. The predicted axial strength based on the models by Furlong (1968), Xiao (1989), Zhong (1985), CAN/CSA, AISC LRFD, and AIJ were 1.429, 2.163, 2.311, 1.628, 1.276, and 1.290 kN, respectively, which correspond to measured-to-predicted strength ratios for C1 of 1.67, 1.1, 1.03, 1.47, 1.87, and 1.85, respectively. It is clear that the design specifications highly underestimate the axial strength of bonded columns. The models by Xiao (1989) and Zhong (1985) provide good prediction of axial strength, while the model by Furlong (1968) underestimates the axial strength.

Unbonded CFST Short Columns

The unbonded CFST short columns showed relatively lower stiffness within the elastic range, in comparison to the bonded ones. The tube in the unbonded system was mainly utilized to provide confinement of concrete, rather than providing axial strength and stiffness. The maximum axial strengths of the unbonded columns were slightly higher than those of the bonded columns.

Mander et al. (1988) proposed a unified stress-strain approach for confined concrete members subjected to axial compressive stresses and lateral confinement pressure, based on the yield strength of the transverse reinforcement. The model can be applied for CFST, where the tube is assumed to be yielding in the hoop direction. The model predicts the maximum compressive strength of the confined concrete, which can be used to predict the ultimate axial load, based on the confined concrete core contribution only. The predicted maximum axial load for the unbonded specimens C4 and C5 loaded through the concrete core only is 2,010 kN, which corresponds to measured-to-predicted strength ratio of 1.26. The model underestimated the axial strength since the steel tube has contributed a small axial load resistance as a result of imperfections, despite the attempt to release the tube completely from any contribution in the axial direction. This is evident from the small axial strain measured in the steel tube of C4, (0.0012) as shown in Fig. 6. If the axial load contribution of the tube, associated with the measured 0.0012 axial strain (348 kN), is accounted for in the prediction, the predicted total axial strength would be 2,358 kN, which compares well with the measured average axial strength of C4 and C5, 2,527 kN.

Beam-Column Specimens

Test results indicate similar flexural behavior and failure modes of all tested CFST beam-column specimens and an insignificant ef-
effect of the bond and end loading conditions. Predictions of the axial load–bending moment interaction diagrams for the tested beam-column specimens using the different design codes including the CAN/CSA, AISC LRFD, and the AE are shown in Fig. 12. The test results of the beam-column and short column specimens are also plotted in Fig. 12. The figure clearly indicates that the codes are extremely conservative. This is attributed to the fact that the confinement effect on the concrete core is excluded in design codes. It is also possible that the strain hardening of the steel tube, which is not accounted for in design codes, has enhanced the maximum flexural strength of the beam-column specimens.

Analytical models. The full axial load–bending moment interaction diagram of the test specimens has also been predicted using cracked section analysis, accounting for equilibrium, strain compatibility, material stress strain-curves and adopting the layer-by-layer technique for numerical integration of stresses (Kilpatrick and Rangan 1997b). The analyses are conducted for three conditions including (1) unconfined concrete stress-strain curve and elastoplastic steel behavior, (2) confined concrete stress-strain curve based on Mander et al. (1988) and elastoplastic steel behavior, and (3) confined concrete and accounting for strain hardening of the steel. Fig. 13 shows the idealized section and the stress-strain curves. The interaction curves for the three cases are plotted in Fig. 12. The behavior shows that although the experimental moments were slightly underestimated, the strain compatibility model, accounting for both concrete confinement and strain hardening of steel, provides better prediction compared to design specifications. It should be noted however that this model is complex and time consuming.

A simplified model is introduced to provide prediction of the flexural strength $M_u$ of CFST under different axial loads $P_a$, based on superposition of the flexural strengths of the concrete core $M_c$ and the steel tube $M_s$, as given in Eq. (3) and illustrated in Fig. 14:

$$M_u = K(M_c + M_s)$$

(3)

The model accounts for the confinement effect of the steel tube on the concrete core as well as the strain hardening effect in the steel tube, through the factor $K$ of Eq. (3). The strength enhancement factor $K$ is based on the expression developed by Pradesley and Park (1987) for confined concrete columns and is given in the following equation:

$$K = 1.13 + 2.35 \left( \frac{P_a}{f_{c} A_{c}} \right)^{0.1} \text{ for } \frac{P_a}{f_{c} A_{c}} < 0.1$$

$$K = 1.13 + 2.35 \left( \frac{P_a}{f_{c} A_{c}} \right)^{0.1} \left( \frac{P_a}{f_{c} A_{c}} \right)^{0.1} \text{ for } \frac{P_a}{f_{c} A_{c}} > 0.1$$

(4)

where $A_c$ = area of the entire concrete section; $f_c$ = compressive strength of the unconfined concrete; and $P_a$ = externally applied axial load.

The flexural strength of the steel tube $M_s$ is based on the plastic capacity of the tube, as shown in Fig. 14, and given in Eq. (5):

$$M_s = f_c Z_e$$

(5)

where $f_y =$ yield strength of the tube; and $Z_e =$ plastic section modulus of the tube, given as $(D_2^2 - D_1^2)/8$, $D_2$ and $D_1$ = outer and inner diameters of the tube.

The model assumes that the flexural strength of concrete core $M_c$ is based on the American Concrete Institute (ACI) stress block approach, in which the resultant of the stress block, $C_{c_r}$, is equal to the external axial load, $P_a$, as shown in Fig. 14. $M_c$ is calculated from the following equations:

$$M_c = C_{c_r}$$

(6)
C_t = 0.85f_c' A_t

(7)

where $e$ = eccentricity of the compression force $C_t$; and $A_t$ = area of the concrete segment in compression.

Given the external axial load $P_e$, equal to the total compressive force in concrete $C_t$, Eq. (7) can be used to determine $A_t'$. Using the geometry of the circular segment $A_t'$, the eccentricity $e$, which is the distance between the center of geometry of the segment and the center of the full circle, can be determined. Fig. 15 is provided for convenience to determine the eccentricity $e$ directly for any axial load $P_e$. Eqs. (3) to (6) are then used to predict the moment resistance $M_t$ at any axial load $P_e$ as shown in Fig. 12. It should be noted that this set of equations is only valid up to an axial load level $P_e$ equal to 0.85$f_c'$, represented by point (a) on the chart in Fig. 12. The corresponding moment resistance at this axial load level $M_t$ is 2.45$M_t$. In order to predict the moment resistance at axial load levels higher than 0.85$f_c'$, interpolation can be used, based on a straight line assumed to connect point (a) to points (b), which represents the pure axial load strength in Fig. 12. According to Mander et al.'s model (1988), Fig. 12 shows the predicted interaction curve based on the proposed model provides reasonable agreement with the measured data and the strain compatibility model. Fig. 8 shows the predicted lateral load resistance based on the flexural strength of the beam-column specimens using the simplified model and the different specifications. The proposed model provides good agreement, while the design specifications significantly underestimated the lateral load resistance. In Fig. 8, the predicted lateral load resistance decreases as the lateral deflection increases to account for the $P-\Delta$ effect, in which, the constant axial load produces secondary moments.

Conclusion

The behavior of CFST has been studied under axial compression and combined axial compression and lateral cyclic loads. The experimental program was conducted on tubes with a relatively small diameter (152 mm), therefore, the conclusions may or may not be valid for large diameter tubes. Further research is also still needed on CFST with actual end connections used in the field. Based on the experimental and analytical investigations, the following conclusions are drawn.

1. The axial strength capacity of CFST short columns occurred at axial strain ranges from 0.009 to 0.012. The strength was 65 to 75% higher than the strength of the composite section based on unconfined concrete strength.
2. The behavior of short CFST columns after cracking of concrete was very ductile. The load dropped to a sustained residual strength, approximately equal to the strength of the composite section based on unconfined concrete strength, and at least 6% axial strain was reached.
3. The maximum and residual axial load capacities of unbonded CFST short columns were slightly higher than those of bonded specimens, due to confinement effect. However, stiffness of unbonded columns was slightly lower due to the absence of contribution of the steel tube in the axial direction.
5. The behavior of CFST beam columns under combined constant axial compression and lateral cyclic loads was very ductile. The specimens maintained their flexural strength during the elastic range. Failure due to local buckling of the steel tube occurred at lateral deflection of about 3.5 times the yielding deflection.
6. The bond and end loading conditions had insignificant effect on flexural strength of CFST beam columns, however, unbonded specimens yielded at higher moment than that of bonded ones.
7. Bonded CFST beam columns exhibited better ductile behavior than unbonded specimens, which suffered rapid flexural strength deterioration after initiation of local buckling of the tube.
8. Both unbonded and bonded beam columns failed by fracture of the steel tubes in the local buckling zone as a result of successive buckling and straightening.
9. The available design specifications significantly underestimated the maximum axial capacity of short CFST columns as well as the flexural strength of CFST beam columns subjected to axial compression and bending.
10. The proposed simplified model provides good prediction of the flexural strength of beam columns under a given axial compression load.

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Notation

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
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<tbody>
<tr>
<td>$A_r$</td>
<td>area of the total cross section of the concrete core;</td>
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<tr>
<td>$A_t'$</td>
<td>area of the concrete segment in compression;</td>
</tr>
<tr>
<td>$C_t$</td>
<td>the resultant compression force in the segment of the concrete core under compression;</td>
</tr>
<tr>
<td>$D_t$</td>
<td>inner diameter of the steel tube;</td>
</tr>
</tbody>
</table>
$D_o$ = outer diameter of the steel tube; 
$E$ = Young’s modulus of steel;
$e$ = eccentricity of the compression force $C_p$, measure to the center of the circle;
$f'_c$ = compressive strength of unconfined concrete; 
$f_y$ = yield strength of the steel tube; 
$K$ = strength enhancement factor; 
$M_c$ = contribution of the concrete core to the flexural strength of the CFST beam-column; 
$M_s$ = flexural strength of CFST at any given axial compression load; 
$M_p$ = plastic moment capacity of the steel tube; 
$P_L$ = axial compression load acting on CFST beam-column; 
$P_y$ = the load that produces yield in the steel tube; 
$R$ = inner radius of the steel tube; 
$t$ = wall thickness of steel tube; 
$Z_p$ = plastic section modulus of the steel tube; 
$\Delta_0$ = lateral deflection of CFST beam-column specimen at yielding; 
$\sigma_y$ = tensile stress in the steel tube in the hoop direction; 
$\sigma_x$ = compressive stress in the steel tube in the axial direction; 
$e_y$ = tensile strain in the steel tube in the hoop direction; 
$e_x$ = compressive strain in the steel tube in the axial direction; 
$v$ = Poisson’s ratio of steel.

References
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