Numerical Analysis of Concrete-Filled Circular Steel Tubes

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**Recommended Citation**

Deng, Yaohua; Norton, Terri R.; and Tuan, Christopher Y., "Numerical Analysis of Concrete-Filled Circular Steel Tubes" (2012). Civil Engineering Faculty Publications. 9.
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Concrete-filled steel tubes have been widely used in building and bridge construction. In this paper, the flexural behaviour of concrete-filled steel tubes is investigated using finite-element analysis and theoretical sectional analysis. The numerical results from both analytical methods are validated against published experimental test results. It is shown that both methods are capable of predicting the elastic and the ultimate moment capacities of concrete-filled steel tubes. Due to its computational efficiency, theoretical sectional analysis is more suitable for the design of concrete-filled steel tubes than finite-element analysis.

1. Introduction
Concrete-filled steel tubes (CFTs) are composite members possessing the favourable attributes of both concrete and steel. The continuous confinement provided to the concrete core by the steel tube enhances the core strength and ductility. The concrete core restrains inward buckling of the steel tube, while the steel tube serves as tensile reinforcement for the concrete.

Studies on circular CFT beam–columns under combined flexural and axial loads have been conducted by many researchers (e.g. Fujimoto et al., 2004; Inai et al., 2004; Sakino et al., 2004). Because bending is a special state of beam–column design and is needed to construct the axial load–moment interaction diagram, the flexural behaviour of CFTs has been investigated extensively in the literature (Elchalakani et al., 2001; Han, 2004; Han et al., 2006; Wheeler and Bridge, 2006). Tuan (2004) developed post-tensioned CFTs (PTCFTs) by post-tensioning the concrete core of the CFT; these were used for the construction of the bottom chords in a pedestrian arch bridge in Aurora, Nebraska (Figure 1). To fabricate the PTCFT members, a post-tensioning strand (or a threaded bar) was positioned inside a steel tube, using spacers to ensure that the tension strand was placed at the centre along the steel tube (Figure 2). A moderate post-tension was applied to the concrete core after it had reached the required strength. Experimental results and numerical analysis (Deng et al., 2011) showed that the PTCFTs had a higher flexural strength than CFTs that were not post-tensioned.

Finite-element analysis (FEA) has been used by many researchers to study the behaviour of CFT beams or beam–columns under bending. Susantha et al. (2001) presented a three-dimensional FEA methodology for a quantitative evaluation of confinement in concrete-filled box-shaped, unstiffened steel columns. The confinement effects of concrete in non-circular sections was assessed in terms of maximum average lateral pressure. Hu et al. (2005) proposed material constitutive models for CFT beam–columns subjected to combinations of axial compressive force and bending moment using Abaqus software. The numerical results compared well with the experimental data. Lu et al. (2009) used FEA to
In this paper, the flexural behaviours of non-post-tensioned and post-tensioned CFT beams are investigated numerically using FEA and theoretical sectional analysis (TSA). The theory and procedures of FEA and TSA are described in detail. To validate the FEA and TSA models, the predicted moment capacities, load-deflection relationships and strains in the steel tubes are compared with experimental results reported in the literature.

2. Finite-element analysis
Based on the American Institute of Steel Construction design manual (AISC, 1999), the diameter-to-thickness ratio of the steel tubes used in the tests should meet the limit for compact circular hollow sections in flexure

\[ (D/t) < 0.31E/f_y \]

where \( D \) is the outer diameter of the steel tube, \( t \) is the thickness of the steel tube, \( E \) is the modulus of elasticity of steel and \( f_y \) is the yield strength of steel.

Equation 1 implies that the hollow steel tubes would only attain the yielding moment (but not plastic moment) without local buckling. However, local buckling is significantly restrained by the concrete infill in CFTs. For instance, Bradford et al. (2002) proposed a slenderness limit for CFTs

\[ \lambda_s = (D/t)(f_y/250) \leq \lambda_{sy} = 125 \]

where \( \lambda_s \) is the section slenderness of the steel tube and \( \lambda_{sy} \) is the section slenderness limit of the steel tube. Bradford et al. (2002) stated that a circular steel section would reach post-buckling yield strength, if the \( D/t \) ratio of the circular tube satisfied Equation 2. All the selected specimens in this paper satisfy this slenderness limit of 125. In the analyses, the maximum strength of steel is set equal to the yield strength. Poisson’s ratios of concrete and steel are assumed to stay constant at 0.2 and 0.31, respectively.

2.1 Constitutive relationship of the steel tube
The elastic-perfectly plastic uniaxial material model is used for the steel tube with an elastic modulus of 200 GPa. The steel tube material is assumed to follow the von Mises yield criterion and the associated flow rule after yielding.

2.2 Constitutive relationship of the concrete
The Drucker–Prager (DP) plasticity model is employed for confined concrete as a pressure- and constraint-sensitive material. This is a modification of the von Mises yield criterion, which accounts for the influence of the hydrostatic stress component: the higher the hydrostatic stress (confinement pressure), the higher the yield strength. The yield surface does not change with progressive yielding, hence there is no hardening rule and the material is elastic–perfectly plastic. However, an increase in the strength of concrete is allowed as long as the hydrostatic stress...
increases. This yield surface is a circular cone with the material parameters chosen such that it corresponds to the outer apices of the hexagonal Mohr–Coulomb yield surface (Ansys, 2007).

The yield criterion is given by

\[ F = 3\beta\sigma_m + \frac{1}{2} \{s\}^T [M] \{s\}^{1/2} - \sigma_y = 0 \]

where \(\sigma_m\) is the mean or hydrostatic stress (i.e. \((\sigma_x + \sigma_y + \sigma_z)/3\)), \(\{s\}\) is the deviatoric stress

\[[M]_{6\times6} = \left[ \begin{array}{cccc} I_{3\times3} & 0 \\ 0 & 2I_{3\times3} \end{array} \right]\]

where \(I_{3\times3}\) is the \(3 \times 3\) identity matrix and \(\beta\) is a material constant given by

\[ \beta = \frac{2 \sin \phi}{3^{1/2}(3 - \sin \phi)} \]

where \(\phi\) is the angle of internal friction. The material yield parameter \(\sigma_y\) is defined as

\[ \sigma_y = \frac{6c \cos \phi}{3^{1/2}(3 - \sin \phi)} \]

where \(c\) is the cohesion value.

The dilatancy angle \(\phi_t\), is a parameter to determine the flow rule and to control the plastic flow. If \(\phi_t = \phi\), the flow rule is associative, plastic straining occurs normal to the yield surface and there will be a volumetric expansion of the material with plastic strains. If \(\phi_t < \phi\), the flow rule is non-associative, and there will be less volumetric expansion. If \(\phi_t = 0\), there will be no volumetric expansion.

Mirmiran et al. (2000) suggested that the cohesion and angle of internal friction for concrete are related to concrete strength \(f'_c\) by

\[ f'_c = \frac{2c \cos \phi}{1 - \sin \phi} \]

\[ k = \frac{1 + \sin \phi}{1 - \sin \phi} \]

where \(k\) is the confinement effectiveness factor, proposed to be 4-1 by Richart et al. (1928).

2.3 Finite-element modelling

On account of double symmetric geometry, loading and boundary conditions, only one quarter of each specimen was modelled using Ansys software version 11.0 (Ansys, 2007). Steel was modelled using eight-node Solid45 elements, having three translational degrees of freedom at each node. Concrete was modelled by eight-node Solid65 elements, which are similar to the Solid45 elements but with the additional capabilities of cracking (in three orthogonal directions) and crushing. The element has one solid material and up to three rebar materials in three directions. Thus, this element is commonly used to accommodate non-linear material properties. Prestressing strands were modelled using the Link8 element, which is a uniaxial tension–compression element with three translational degrees of freedom at each node with no bending stiffness. An incremental procedure for FE modelling of the CFTs was implemented as follows.

(a) Model the concrete, steel tube in a CFT and prestressing strands (for PTCFT only, if any) with the element types described above.

(b) Input geometric properties such as diameter and thickness of steel tube, length and clear span of specimen, dimension of the strands, etc.

(c) Input the compressive strength of unconfined concrete. The modulus of elasticity and tensile strength of unconfined concrete can be derived, respectively, from Equations 14 and 16 (given later). Shear transfer coefficients of 0.7 and 1.0 are used for an open crack and a closed crack, respectively. The crushing capability is deactivated in the analyses. Also, input the yield strength and modulus of elasticity of the steel tube and strands.

(d) Define the DP parameters; a non-associated flow rule is assumed and the dilatancy angle is set equal to zero.

(e) Generate a mesh of elements for one quarter of the specimen and define the boundary conditions, taking two planes of symmetry into account.

(f) Establish common nodes between the concrete core and the prestressing strand for the PTCFT specimen in this paper, even though the prestress is applied due to post-tensioning. Once the strand is anchored at the end of a CFT beam, strand deformation is concentrated at the midspan of the beam under one-point loading. The initial post-tension of strand is imposed by assigning a temperature drop to the strand elements, which generates the equivalent prestress. The stress–strain curve of the 270 ksi (1862 MPa) strand is available from the PCI design handbook (PCI, 2004)

\[ \varepsilon_{ps} = 0.0086, \quad f_{ps} = 196552 \varepsilon_{ps} \text{ (MPa)} \]
where \( \varepsilon_{ps} \) and \( f_{ps} \) are the strain and the stress in the strand respectively.

(g) For systems with only one-point or two-point loading applied, a direct relationship between force and displacement can be established. Thus, one-point or two-point loading is applied by the displacement control method according to the original test setup and the displacement is applied to the nodes where the load was applied. The CNVTOL command is adopted to set convergence values for the non-linear analysis and L2 norm is selected to check the square root sum of squares (SRSS). The convergence tolerances are set for both displacement and forces. Simply supported boundary conditions are imposed on the node at the clear span locations such that the beam is free to rotate at the supports.

(h) Set parameters to facilitate the convergence of the non-linear problem solutions

(i) regarding keyopts, suppress extra displacement shapes and include tensile stress relaxation after cracking

(ii) set appropriate interval and number of load steps and substeps

(iii) when large plastic deformation develops, larger convergence tolerances for displacements and forces should be used and maximum equivalent plastic strain allowed within a substep (PLSLimit) could be set larger by CutControl command

(iv) open auto-step and predictor to solve potential computational problem

(v) check reasonable element sizes and shapes.

(i) Post-processing: compute the applied load and moment at each section from the reaction force at the support. Plot the time histories of moment, deflection and strains at midspan.

### 3. Theoretical sectional analysis

#### 3.1 Assumptions

(a) Full bond between the concrete core and steel tube is assumed and the CFT beam is analysed in a plane strain condition.

(b) Given that Poisson’s ratios of concrete and steel in the elastic state are 0·2 and 0·31, respectively, it is assumed that Poisson’s ratio of concrete \( \nu_c \) varies linearly from 0·2 to 0·31 when concrete strain increases from 0 to 0·001, and from 0·31 to 0·5 when concrete strain increases from 0·001 to the strain at the maximum strength of confined concrete (e.g. 0·005). After the concrete reaches maximum strength, Poisson’s ratio is assumed to be 0·5.

(c) No local buckling is considered in the algorithm and the maximum strength of steel is set equal to the yield strength.

#### 3.2 Constitutive relationship of the steel tube

The steel tube is assumed to follow the uniaxial elastic–perfectly plastic material model with an elastic modulus of 200 GPa. Compared with axial and hoop stresses in the steel tube, the radial stress is negligible. Therefore, a biaxial stress state is adopted in the algorithm and the von Mises yield criterion is applied for the strength of steel. To determine the plastic strain of the steel tube, it is assumed to follow an associative flow rule after yielding.

#### 3.3 Constitutive relationship of the concrete

On average, the 102 mm × 204 mm cylinder strength of concrete is 80% of the 150 mm cube strength (Unesco, 1971). The cube strength of concrete should be converted into cylinder strength before using the algorithm.

#### 3.3.1 Confined concrete

Many researchers have investigated the effects of confinement on the compressive strength and ductility of concrete compression members. Knowles and Park (1970) summarised these efforts and found a simple relation between the increase in concrete strength and lateral confined pressure

\[
f_{cc} = f^c + k f^l
\]

where \( f_{cc} \) is the axial compressive strength of confined concrete, \( f^c \) is the uniaxial compressive strength of unconfined concrete, \( f^l \) is the lateral confining pressure and \( k \) is a coefficient related to the concrete mix and lateral pressure. Richart et al. (1928) reported the average value of \( k \) from experimental data to be 4·1.

Mander et al. (1988a, 1988b) proposed a unified stress–strain approach for confined concrete applicable to both circular- and rectangular-shaped transverse reinforcement. The axial compressive concrete stress \( \sigma_{ca} \) is given by

\[
\sigma_{ca} = \frac{f_{cc} x r}{r - 1 + x'}
\]

where

\[
x = \frac{\varepsilon_c}{\varepsilon_{cc}}
\]

in which \( \varepsilon_c \) is the longitudinal compressive concrete strain and

\[
\varepsilon_{cc} = \varepsilon_c' \left[ 1 + 5 \left( \frac{f_{cc}}{f^c} - 1 \right) \right]
\]

where \( f_{cc}^c \) and \( \varepsilon_{cc} \) are the unconfined concrete strength and corresponding strain, respectively (generally \( \varepsilon_{cc} = 0·002 \) can be assumed), and
13. \[ r = \frac{E_c}{E_c - E_{sec}} \]

where the elastic modulus of concrete can be expressed as \( \text{Wang et al., 2007} \)

14. \[ E_c = 4700(f'_c)^{1/2} \text{ (MPa)} \]

and

15. \[ E_{sec} = f'_{ec} e_{ec} \]

3.3.2 Tensile stress–strain relationship of the concrete

The tensile strength \( f'_t \) (in MPa) of the concrete is related to its compressive strength \( f'_c \) (in MPa) by \( \text{Wang et al., 2007} \)

16. \[ f'_t = 0.56(f'_c)^{1/2} \]

and the maximum tensile strain associated with \( f'_t \) is calculated by

17. \[ \varepsilon_t = 1.8 \frac{f'_t}{E_c} \]

The maximum tensile strain concrete can sustain is about 0.0002. The stress–strain relationship for concrete under tension was proposed by Shah and Gopalaratnam (1985) as

18. \[ \sigma = -f'_t \left( \frac{\varepsilon}{\varepsilon_t} \right)^2 + 2f'_t \left( \frac{\varepsilon}{\varepsilon_t} \right) \]

3.4 Deformation compatibility of compression zone

The confining pressure on the compression zone of the concrete core due to axial compression may be determined by using the compatible deformations at the concrete and steel tube interface. The deformation compatibility at the concrete and steel interface (i.e. \( r = r_c \)) is given by \( \text{Deng et al., 2011} \)

19. \[ \frac{\nu_c E_{ec}}{E_c} - \sigma_t \left( \frac{1 - \nu_c}{E_c} \right) = \frac{\sigma_t (D - 2t)}{2E_t} \]

The confining pressure \( \sigma_t \) produced by axial compressive stress on the compression zone can, therefore, be readily determined from Equation 19.

3.5 Implementation of the numerical algorithm

A numerical algorithm was developed for predicting the moment capacities of circular CFTs. The concrete core and the steel tube in a cross-section are each divided into a large number of horizontal layers parallel to the neutral axis. The area of each layer of the concrete core and the steel tube is calculated based on the geometry of the cross-section. The stresses in the concrete and steel layers are determined using their respective constitutive models. The influences on the strength or ductility of concrete and steel due to the composite actions are taken into account. Since the neutral axis location \( y_{na} \) is not known a priori for a given curvature, an iterative procedure is implemented to find the location to yield zero net axial force. Once the neutral axis solution is converged, the strain in each layer and the bending stresses in the concrete and the steel layers are determined using their respective constitutive models. The axial force can be obtained by summing the axial force of each layer. The moment acting on the cross-section is subsequently calculated by summing the constituent moments produced by all the layers, about the neutral axis.

3.6 Numerical analysis of post-tensioned concrete-filled circular steel tubes

The algorithm was further modified to include post-tensioning effects. An initial compressive strain (or stress) was added to the strains (or stresses) induced in the concrete under flexure. The concrete strength was enhanced due to the confining pressure, which, in turn, was induced by the post-tensioning force. For concrete in the tension zone, the initial compressive strains must be offset by flexure before the concrete stresses become tensile. Including post-tensioning stress, the neutral axis location \( y_{na} \) was changed and could be determined by the location of zero strain. The post-tensioning tendon was assumed to be at the centre of the section. Although the initial post-tensioning stress in tendon was the jacking stress, the magnitude of the prestressing stress should change along with the strain of the centre fibre of the section. The post-tensioning force was included in the net axial force and moment calculations.

3.7 Numerical analysis of non-linear moment–deflection relationship

After the moment–curvature relationship of the cross-section has been derived from the above analysis, the incremental procedure for deriving the load–deflection relationship is described by \( \text{Deng et al.} (2011) \).

4. Validation of FEA and TSA against the flexural testing results

Based on the procedures described above, FEA and TSA were performed to investigate the flexural behaviour of CFT and PTCFT beams. Furthermore, both FEA and TSA methods were verified against flexural testing results reported in the literature \( \text{Deng et al., 2011; Han et al., 2006; Wheeler and Bridge, 2006} \).
4.1 Published test results

The literature reports on flexural tests conducted by one-point and two-point loading methods, as summarised in Table 1 and Figure 3. The shear span $a$ and clear span $L_0$ are depicted in Figure 3.

Deng et al. (2011) tested one circular CFT beam and one post-tensioned circular CFT beam (PTCFT) made from the same circular steel tube. The initial applied pre-stressing force was 145 kN, around 75% of the ultimate strength of strands. Typical seven-wire, low-relaxation, 0.5 in. (13.7 mm) nominal diameter prestressing strands were used for post-tensioning. These beams, simply supported, were tested by one-point loading at midspan. Strain gauges and displacement transducers were used to measure the beam deflections and strains, respectively, as shown in Figure 3.

The beam segment near midspan gradually sustained large plastic deformation and the bottom of the steel tube eventually ruptured, exposing concrete spalls from tensile failure, as evident in Figure 4(a). The tensile cracks were concentrated in the midspan region of PTCFT219 and most of the cracks extended into the compression zone, as shown in Figure 4(b). Crushing failure of the concrete in the compression zone was also evident as the midspan deflection exceeded 300 mm (Figure 4(c)). Due to the good ductility of the CFT beams, as suggested by Lu et al. (2009), the deflection at midspan can approach one tenth of the length of the specimen. In this analysis, the ultimate moment is defined as the moment when the deflection at midspan reached $L_0/20$ of the specimen (i.e. 155 mm). The beneficial effects of post-tensioning on the flexural strength of a concrete-filled circular steel tube were observed in the bending tests. Due to post-tensioning, the moment capacity of PTCFT219 was larger than that of CFT219 (92.4 kN m and 87.6 kN m, respectively), as shown in Table 1.

Han et al. (2006) tested a total of 36 CFT beam specimens with varying parameters of sectional type, steel yielding strength, concrete strength, diameter to thickness ratio of the tube and shear span to depth ratio. Two specimens with the same shear span to depth ratio were tested under one-point loading and two-point loading, respectively. The specimens were labelled as CB$i$-$j$ ($i = 1, 2, 3, 4, 5, 6, j = 1, 2$), where $i$ denotes the different shear spans and $j$ denotes the different specimens with an identical shear span. Han et al. (2006) defined the ultimate moment of the CFT beams as the bending moment when the maximum fibre strain of the section reached 0.01. The ultimate moments of the specimens are listed in Table 1.

Wheeler and Bridge (2006) tested four specimens of two different sizes. The CFT beam specimens were simply supported and loaded by two-point loading with identical shear span.

4.2 FEA assumptions

Initially, rigid to flexible contact was assumed between the concrete core and the steel tube since the stiffness of the concrete is significantly smaller than that of the steel tube. Surface-to-surface contact elements were specified at the interface between the concrete core and the steel tube, with Target170 on the inside steel tube and Conta173 on the outside of the concrete core. However, the FEA model yielded a small axial stress in the concrete and noticeable slippage during loading. In reality, the slippage between the concrete and the steel tube is found to be very small (Deng et al., 2011; Wheeler and Bridge, 2006). Roeder et al. (1999) demonstrated that eccentric loading (i.e. due to moment) significantly increased bond stress capacity. Thus, full bond between the concrete core and steel tube was assumed; that is, the concrete core and the steel tube shared common nodes at the interface.

A mesh refinement analysis was performed and it was found that the computational results were not sensitive to changes in element sizes or meshes. A typical mesh for specimen CFT219 is shown in Figure 5.
<table>
<thead>
<tr>
<th>Specimen</th>
<th>$L_0$: mm</th>
<th>$a$: mm</th>
<th>$D$×$t$: mm</th>
<th>$D/t$</th>
<th>$f_y$: MPa</th>
<th>$f_c$: MPa</th>
<th>$\lambda_s$</th>
<th>Moment capacity: kN m</th>
<th>Maximum $M_{exp}/M_{FEA}$</th>
<th>$M_{exp}/M_{TSA}$</th>
<th>Loading method</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFT219</td>
<td>3100</td>
<td>1550</td>
<td>219 × 3·7</td>
<td>59</td>
<td>340</td>
<td>*58</td>
<td>80·2</td>
<td>87·6</td>
<td>90·1</td>
<td>80·8</td>
<td>3·9</td>
</tr>
<tr>
<td>PTCFT219</td>
<td>2950</td>
<td>1475</td>
<td>219 × 3·7</td>
<td>59</td>
<td>340</td>
<td>*58</td>
<td>80·2</td>
<td>92·4</td>
<td>101·5</td>
<td>89·3</td>
<td>3·9</td>
</tr>
</tbody>
</table>

Deng et al. (2011)

<table>
<thead>
<tr>
<th>Han et al. (2006)</th>
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<tbody>
<tr>
<td>CB1-1</td>
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<tr>
<td>CB1-2</td>
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<td>CB2-2</td>
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<td>CB3-1</td>
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<td>CB5-1</td>
</tr>
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<td>CB5-2</td>
</tr>
<tr>
<td>CB6-1</td>
</tr>
<tr>
<td>CB6-2</td>
</tr>
</tbody>
</table>

Wheeler and Bridge (2006)

| TBP002          | 3800   | 1300   | 406 × 6·4   | 63·4   | 350        | 40         | 89·1      | 489              | 495             | 486            | 3·6           | 0·988         | 1·006         | Two point |
| TBP003          | 3800   | 1300   | 406 × 6·4   | 63·4   | 350        | 55         | 89·1      | 498              | 530             | 501            | 4·1           | 0·939         | 0·994         | Two point |
| TBP005          | 3800   | 1300   | 456 × 6·4   | 71·3   | 350        | 48         | 100·1     | 630              | 658             | 628            | 3·4           | 0·957         | 1·003         | Two point |
| TBP006          | 3800   | 1300   | 456 × 6·4   | 71·3   | 350        | 56         | 100·1     | 647              | 702             | 641            | 3·6           | 0·921         | 1·009         | Two point |
| Mean            | 3800   | 1300   | 456 × 6·4   | 71·3   | 350        | 56         | 100·1     | 647              | 702             | 641            | 3·6           | 0·97           | 1·10          |            |

* Cube concrete strength

Table 1. Comparison of numerical results with experimental data: $L_0$, clear span; $a$, shear span; $D$, tube diameter; $t$, tube thickness; $f_y$, yield strength of steel; $f_c$, concrete strength; $\lambda_s$, section slenderness
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4.3 Comparison of FEA, TSA and experimental results
The ultimate moment capacities predicted by FEA and TSA are compared against the experimental results in Table 1. Good agreement was obtained using FEA with a mean ratio $M_{\text{exp}}/M_{\text{FEA}}$ of 0.97 and a coefficient of variance (CoV) of 0.06; TSA achieved a mean ratio $M_{\text{exp}}/M_{\text{TSA}}$ of 1.10 and a CoV of 0.08. For both one-point and two-point loading methods, the ultimate moment capacities predicted by the FEA models are always larger than those predicted by the TSA models. This is due to the fact that the DP model of FEA ignores the softening of concrete strength and overestimated the moment capacities. The ultimate moment capacities predicted by TSA are smaller than the experimental results because the strain-hardening effect of the steel is not taken into account in the numerical algorithm. The predicted moment–deflection relationships of specimens CFT219 and PTCFT219 of Deng et al. (2011) at midspan are compared with the experimental results in Figures 6 and 7, respectively. The
predicted moment–deflection relationships of specimens CB1-1, CB1-2, CB2-1, CB3-1 and CB3-2 of Han et al. (2006) are with the experimental results in Figures 8 and 9. The predicted moment–deflection relationships of TBP002 and TBP005 (Wheeler and Bridge, 2006) are compared with the experimental results in Figures 10 and 11, respectively. The maximum confining pressure for each specimen from the TSA model is given in Table 1. The relationships of confining pressure and average compressive stress in the compression zone of concrete with the curvature for CFT219 are shown in Figure 12.

The deflection shapes of CFT219 approaching failure under bending testing and the FEA simulation are compared in Figure 13. Both deflection shapes clearly exhibit a V-shape, since the beam experienced large plastic deformation at midspan.

For both FEA and TSA, the steel tube yields successively at the bottom, top and middle of the midspan section of CFT219, as depicted in Figure 14. The steel tube begins to yield at smaller moment of midspan section in the TSA model than in the FEA model. The tensile strength of the steel slightly increased and the compressive strength slightly decreased after yielding due to the biaxial stress state.
For specimens CFT219 and PTCFT219, the steel strain–moment relationships before large plastic deformation took place were accurately predicted by both FEA and TSA models (Figure 15). After the steel tube had yielded, the strains at the bottom, top and mid-depth increased significantly in that order. Since the TSA model ignores strain hardening and inelastic local buckling and the FEA model ignores strain hardening and the softening of concrete strength, the strains after the steel yielded could not be accurately predicted. Nevertheless, the strains predicted were in reasonable agreement with the experimental data.

**5. Conclusion and recommendations**

Concrete-filled steel tubes (CFTs) are composite members possessing the best attributes of both concrete and steel. The flexural behaviour of CFTs and post-tensioned CFTs was investigated analytically. Finite-element analysis (FEA) was performed using the elastic–perfectly plastic uniaxial material model for steel and the Drucker–Prager (DP) plasticity model for concrete. Theoretical sectional analysis (TSA) was carried out by dividing the cross-section of a circular CFT member into a large number of horizontal layers parallel to the axis of bending. The elastic–perfectly plastic uniaxial material model was also used for the steel tube and confined concrete theory was applied to the concrete core.

The moment–deflection relationships and strains and stresses in
the steel tube were predicted using TSA and FEA. Both the TSA and FEA models were validated against test results reported in the literature. Generally, TSA gave more conservative results than FEA. The ultimate strength is overestimated by the FEA model due to the fact that the DP model ignores strain softening of concrete. Since TSA ignores strain hardening in the steel and inelastic local buckling and FEA ignores strain hardening and softening of the concrete strength, the strains after significant yielding in the steel could not be accurately predicted. Nevertheless, the predicted strains were in reasonable agreement with published experimental data.

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