NORMAL AND HIGH STRENGTH CONCRETE-FILLED STEEL BOX COLUMNS UNDER AXIAL COMPRESSION

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This manuscript presents the behaviour and strength of axially loaded Concrete-Filled Steel Tube (CFST) columns using normal and high-strength concrete taking to account phenomena of geometry and material nonlinearities. An effective finite element model (FEM) is simulated to carry out the analysis on rectangular CFST columns with considering imperfections and residual stresses which significantly reduce ultimate strengths of columns under axial compression. The study is carried out over a range of concrete strength from 30 to 170 MPa and ratios of outer diameter-to-thickness (D/t) varying between 15 and 80. Deriving from the test results in published studies, an empirical formulas is generated to determine the confining pressure (f_r). FEM is performed using software package ABAQUS with some modifications to declare parameters of Concrete Damaged Plasticity Model (CDPM). Furthermore, the ultimate strengths predicted from FEM are compared with those calculated using the Eurocode EC4.

Keywords: CFST columns; high-strength concrete; confinement effect; concrete damage plasticity model; ABAQUS.

1 Introduction

Nowadays, concrete-filled steel tube columns (CFST) are studied extensively throughout the world because of its high-failure strength and stiffness, excellent ductility, high seismic and fire resistance. It is acceptable that a greater story space can be achieved owing to the high bearing capacity. Furthermore, with advancements of construction technology and material science, the application of high and ultra-high strength concrete (HSC & UHSC) becomes more popular. Rectangular CFST columns have been gained many research interests owing to its conveniences in installation for concrete casting. However, in some cases, the influences of imperfections and residual stresses induced by rectangular steel tube lead to a significant reduction in the strength of CFST columns.

There were many previous studies on CFST columns with the employment of high and ultra-high strength materials such as Han et al. (2005), Liu et al. (2005), Liu et al. (2003), Uy (2001), Sakino et al. (2004), Liu (2005), Liu and Gho (2005), An et al. (2019a, 2019b), An and Fehling (2017a, 2017b). In the nonlinear structural analysis for framed structures, the co-author of this present work, Nguyen and Kim (Nguyen & Kim, 2017, 2018) proposed new plastic hinge approaches for nonlinear analysis of space steel frames considering the flexibility of column-base connections and semi-rigid connections. Uy (2001) carried out experimental tests and

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behavior analysis on CFST columns combining high-strength steel and normal-strength concrete having cylinder strengths (f'_c) from 28 MPa to 32 MPa and very high steel yield stress (f_y) of 817 MPa. . According to Eurocode 4, the compressive strength limit for concrete is $f_c' \leq 60$ MPa and the steel yield strength is $f_y \leq 550$ MPa. In Uy's (2001), the study was conducted by comparing the predicted ultimate strength of EC4 standard with experimental results. The study results show that the ultimate strength using EC4 was overestimated comparing with using the FE model. Liu (2005), Han et al. (2005), Liu and Gho (2005), Liu et al. (2005), Liu et al. (2003), Uy (2001) and Sakino et al. (2004) investigated CFST columns with the employment of high-strength concrete having f_c' from 25 - 85 MPa and high-strength steel having f_y from 262 - 834 MPa. In these studies, the experimental ultimate strengths were also used to evaluate the predictions from EC4, and, geometric imperfections and residual stresses of steel tubes on the compressive behavior of the columns were not considered.

Based on the literature review as mentioned above, this paper is aimed at developing an accurate finite element model (FEM) to estimate the behavior and ultimate strength of CFST columns using high-strength materials. A new formula to estimate the confining pressure is established using experimental data of CFST columns. Moreover, a parametric study employed the proposed formula is investigated to evaluate the guideline EC4 for CFST columns using high-strength steel. For CFST columns using steel circular tubes, Pham and Nguyen (2020) proposes successfully a finite element modelling for axially loaded concrete-filled steel circular tubes published at CIGOS 2019, Innovation for Sustainable Infrastructure.

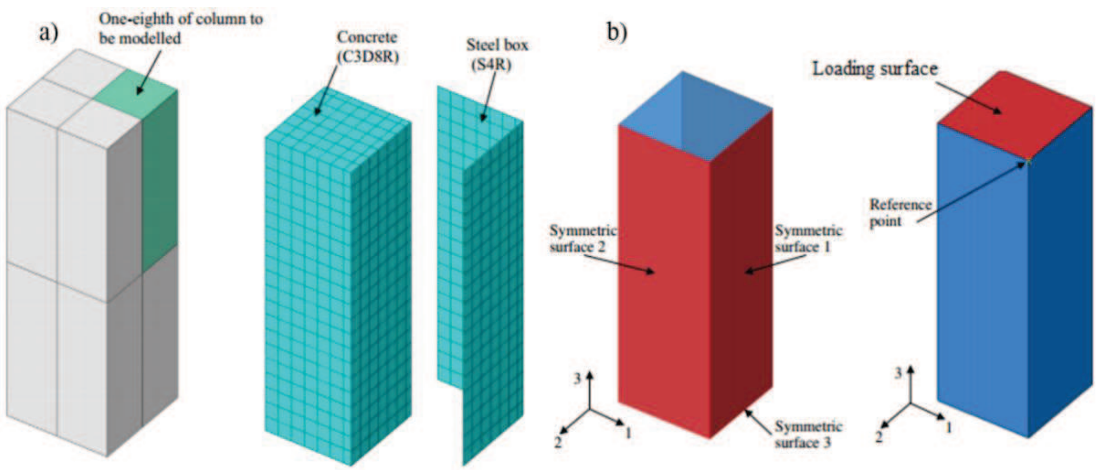

Figure 1. a) Element types and Meshing CFST columns, b) Contact and loading conditions

2 Finite Element Modeling

The steel box in rectangular CFST columns is simulated by S4R shell elements, while the concrete is simulated by solid elements (C3D8R) as shown in Fig. 1a. Deriving from the mesh convergence studies, the mesh size is chosen as B/15, B is the width of steel tubes. The symmetric conditions are adapted on surfaces at the symmetric planes, as illustrated in Fig. 1b. Rigid body constraints are used to tie the end surface of the columns to the reference point located at the center of the end surface of columns. The surface loading is assigned through the reference point.

The contact between concrete core and steel is simulated by the surface-to-surface integrated in ABAQUS. The "hard contact" of the interface is adopted for normal direction, which allows the separation of the interface under tension and no penetration under compression. A penalty function with friction coefficient proposed by Mohr Coulomb model is used for tangential direction. The friction coefficient between steel and concrete is taken as 0.25.

2.1 *Constitutive model of steel*

In this model, a bilinear model is adopted to analyze the steel tube as shown in Figure 2a. Young's modulus is 200 GPa, and Poisson's ratio is 0.3, while the strain hardening modulus is $0.005E_s$.

Box steel sections are easily deformable and sensitive due to local buckling effects by slender, residual stresses, and geometric imperfections. A local imperfection of $B_{eq}/1000$ of the cross-section width which was indicated by Degee et al. (2008) on welded box sections is shown in Figure 2b. Figure 2c demonstrates the residual stress of box steel. The value of the compressive residual stress of about 10% of the steel yield stress is assumed according to Uy (1998).

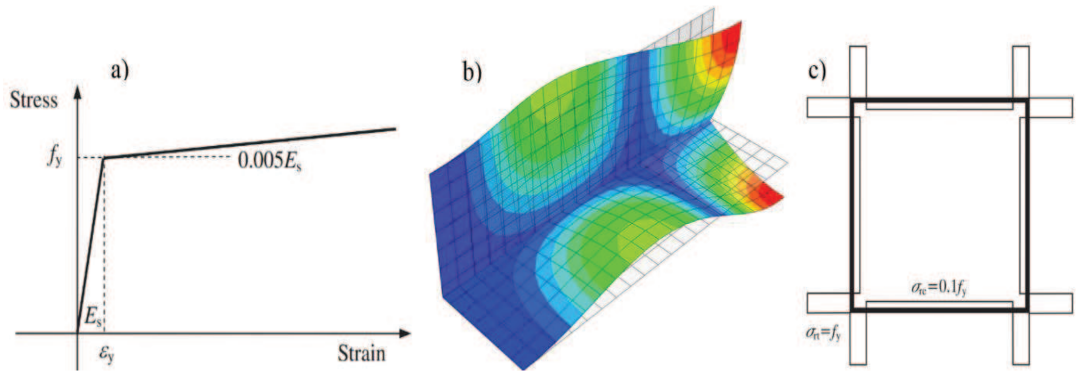

Figure 2. a) Stress–strain curve for steel, b) First buckling mode shape, c) Assuming residual stress of steel

2.1 *Constitutive model of concrete*

Some proposed formulations to represent the concrete behavior consist of two-stages as shown in Figure 3. The ascending branch represents the strain-hardening rule. To describe accurately the concrete behaviour in the ascending stage, Mander et al. (1988) proposed as follows:

$$\frac{f}{f_o} = \frac{(\varepsilon/\varepsilon_0)\times r}{r-1+(\varepsilon/\varepsilon_0)^r};\ r = \frac{E_c}{E_c-(f_0/\varepsilon_0)} \tag{1}$$

where E_c is the elastic modulus of concrete calculated by $E_c = 4700\sqrt{f_c^{'}}$ MPa. In addition, f_c' and ε_c are the peak compressive strength and its corresponding strain of the unconfined concrete, respectively, while f_0 and ε_0 are the peak stress and its corresponding strain of the confined concrete, respectively. The strain at the peak compressive strength of the unconfined concrete ε_c is calculated using Eq. (2) proposed by Tasdermir et al. (1998) and the peak stress f_0 and corresponding strain ε_0 of the confined concrete is calculated using Eqs. (3-4) proposed by Xiao et al. (2010).

$$\varepsilon_c = \left(-0.067\times\left(f_c^{'}\right)^2 + 29.9\times f_c^{'} + 1053\right)\times 10^{-6} \tag{2}$$

$$\frac{f_0}{f_c^{'}} = 1 + 3.25 \times \left(\frac{f_r}{f_c^{'}} \right)^{0.8} \tag{3}$$

$$\frac{\varepsilon_0}{\varepsilon} = 1 + 17.4 \times \left(\frac{f_r}{f_c^{'}} \right)^{1.06} \tag{4}$$

where f_r is defined as the confining stress. There is no confining stress in the elastic stage but the confining stress significantly increases when the steel yields. Beyond the ultimate strength, the value of residual stress f_r becomes constant or tends to increase slighly depending on the value of the confinement index ξ_c. Using regression analysis from 64 specimens, the confining stress f_r is obtained as:

$$f_r = \frac{0.0645 + 0.11 \times f_y}{\left(f_c^{'}\right)^{0.4}} \times \exp\left(-0.07 \times \frac{B_{eq}}{t} \right) \tag{5}$$

With the equivalent width of $B_{eq} = \sqrt{\frac{B^2 + D^2}{2}}$. Besides, the descending branch is simulated by an exponential function suggested by Binici (2005). The residual stress f_{re} of confined concrete and the softening branch reflected by values of α and β are proposed by Tao et al. (2013):

$$f = f_{re} + \left(f_0 - f_{re}\right) \times \exp\left[-\left(\frac{\varepsilon - \varepsilon_0}{\alpha} \right)^{\beta} \right] \tag{6}$$

Using ABAQUS, the Concrete Damaged Plasticity model (CDPM) is utilized to trace the concrete failure with main parameters such as the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian (K_c), flow potential eccentricity (e), dilation angle (ψ), ratio of the compressive strength under biaxial loading over uniaxial compressive strength (f_{bo}/f_c'). These input parameters in this study are determined based on Tao et al. (2013).

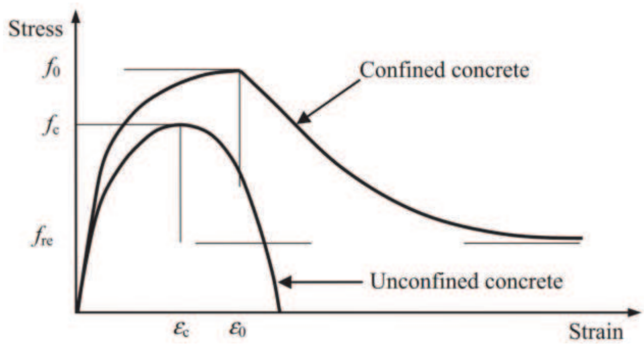

Figure 3. Stress–strain curve of concrete

3 Verification

The material properties and parameters of the specimens are summarized as shown in Table 1. These parameters are adopted for the FE model. The limitations on these parameters of the rectangular specimens are as follows: f_y = 282 – 779 MPa, $f_c{}'$ = 55.3 – 157.54 MPa, H = 60 – 250 mm, H/B = 1 – 1.8, H/t = 12 – 49.46, L/B = 2.98 - 4.54. The ultimate strengths (N_{uc}) predicted by the proposed method and Tao et al. (2013) are compared with the experimental ultimate strengths (N_{ue}) taken from the previous tests as shown in Figure 4. Besides, the ratios of N_{ue}/N_{uc} are also plotted together with the confinement factor ξ as illustrated in Figure 4. Table 2 presents the mean value and standard deviation of N_{ue}/N_{uc} for specimens with different cross-sections. It is observed that the FE model and Tao et al. (2013) slightly underestimate the ultimate strength, while EC4 gives a significantly underestimation of the ultimate strength about 10.4% comparing with the experimental test. Among them, the FE model gives the closest prediction as compared to the test results. It can be seen that the proposed method predicts accurately the ultimate strength of the rectangular CFST columns using high-strength materials.

Table 1. Specimen data of rectangular CFST columns

Source	*Number of tests*	*B* (mm)	*H* (mm)	*t* (mm)	f_y (MPa)	$f_c{}'$ (MPa)
Liu et al. (2003)	05	98.2 - 181.2	100.3 - 182	4.18	550	56.6 - 65.7
Liu & Gho (2005)	10	100 - 130	120 - 180	4.0 - 5.8	300 - 495	56 - 92
Han et al. (2005)	02	60	60	1.87	282	81
Chen et al. (2018)	07	100.3 - 100.7	100.3 - 100.7	2.07 - 7.6	306.7 - 371.6	59 - 130.8
Liew et al. (2016)	13	150	150	8 - 12.5	446 - 846	54 - 62
Xiong et al. (2017)	13	150	150	8-12.5	446-779	142.08-164.1
Liu (2005)	10	108 - 140	108 - 160	4	495	58.40 - 77.60
Khan et al. (2017)	01	109.6	109.6	4.93	762	100
Du et al. (2016a)	02	100 - 120	120 - 180	5.70 - 7.69	423.20 - 514.53	55.3
Du et al. (2016b)	01	150	152	8.28	488.38	55.3

Table 2. Comparison of N_{ue}/N_{uc} between FE predictions and EC4

FE model	SD	Mean
Proposed	0.045	1.005
Tao et al. (2013)	0.057	1.057
EC4	0.050	1.104

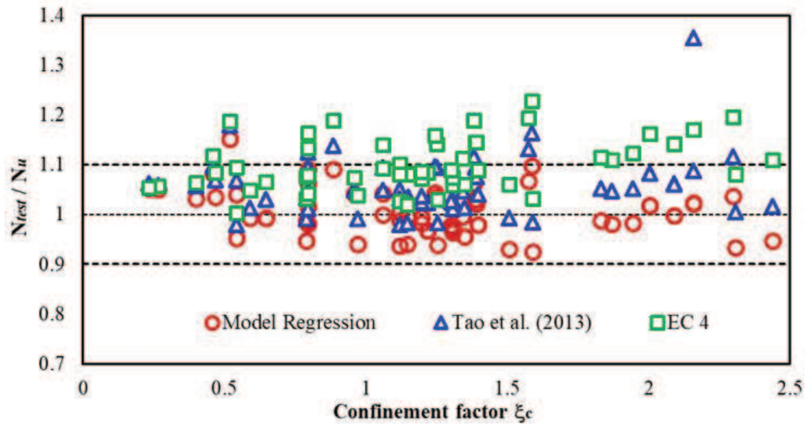

Figure 4. Comparing the results of EC4 and FE models

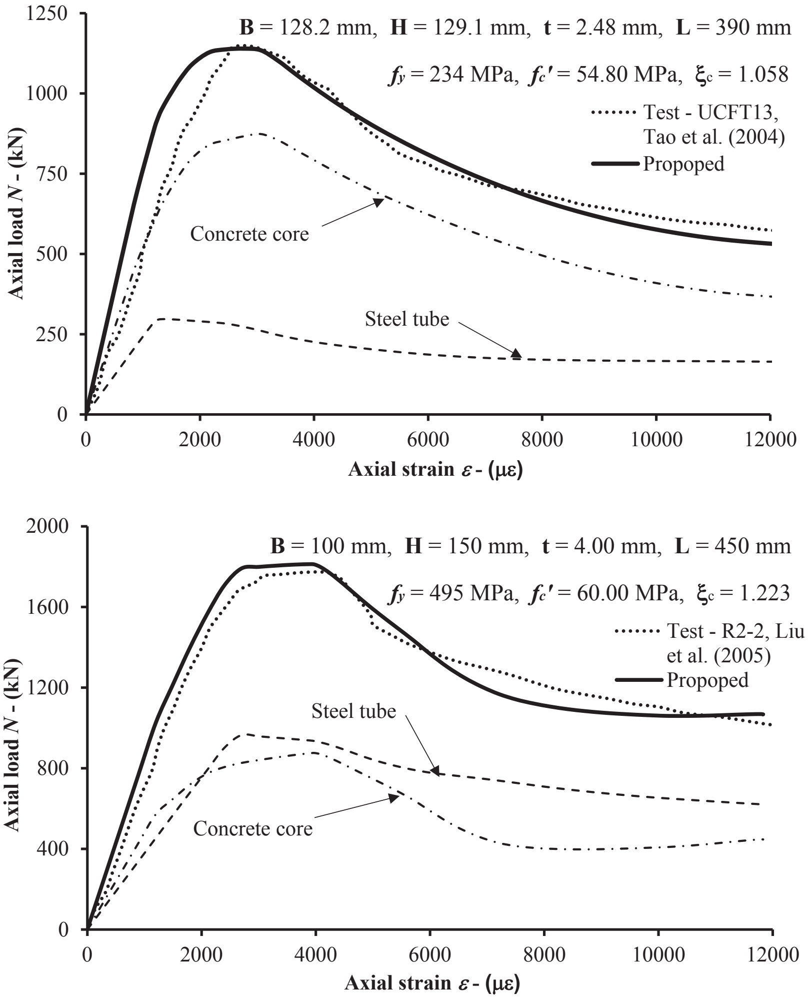

Figure 5. The N-ε curve for two specimens

Figure 5 demonstrates the results of the axial load strain (N-ε) curves between test results and FE results, and also the case of only concrete core or steel tube. The specimens for this comparison include: (1) specimen "UCFT13" tested by Tao et al (2004); (2) specimen "R2-2" tested by Liu et al (2005) . It is found from Figure 5 that the curves obtained from FE models are very identical with those measured by the experimental tests. Furthermore, contributions of the steel tube and concrete core to the ultimate strength are also illustrated by the proposed FE model.

4 Conclusion

The proposed FE model gives the reasonably accurate predictions, thus it is suitable to be adopted for the simulation of the rectangular CFST columns using high-strength materials. The presented FE model takes into account imperfections and residual stresses of steel box, and the confinement effect. The verification examples prove that the presented FE model can predict and trace accurately the ultimate strength and the axial load – strain curves of high-strength CFST columns. As a result, EC4 standard can be used safely to estimate the strength of high-strength CFST columns since it underestimates the ultimate strength of rectangular CFST columns.

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