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Behaviours of circular CFDST with stainless steel external tube: Slender columns and beams

Hui Zhao ^a, Rui Wang ^{a,*}, Dennis Lam ^b, Chuan-Chuan Hou ^c, Rong Zhang ^d

a School of Civil Engineering, Taiyuan University of Technology, Taiyuan 030024, China

b Faculty of Engineering and Informatics, University of Bradford, Bradford BD7 1DP, UK

c School of Transportation Science and Engineering, Beihang University, Beijing 100191, China

d Department of Mechanical Engineering and Automation, Harbin Institute of Technology (Shenzhen), ShenZhen 518055, China

1 **Abstract**

2 This paper presents the experimental and numerical studies on the performance of
3 circular concrete filled double steel tubular (CFDST) slender columns and beams with
4 external stainless steel tube. Twenty-four specimens, including 18 slender columns and
5 6 beams were tested to obtain the failure patterns, load versus deflection relationships
6 and longitudinal strain developments in the stainless steel tube. Finite element (FE)
7 models were established and verified by test results. The validated FE models were then
8 employed to investigate the influences of key parameters, including hollow ratio,
9 eccentric ratio and material strength, on the load-bearing capacity. The load distribution
10 among the components and contact stress between sandwiched concrete and steel tubes
11 were also analyzed. Finally, the design methods for CFDST and hollow CFST members
12 with external carbon steel tube respectively suggested by Han et al. (2018) and Chinese
13 GB 50936-2014 (2014) were employed to evaluate their applicability for the circular
14 CFDST slender columns and beams with outer stainless steel tube.

15 **Keywords:** Concrete filled double steel tubular; Stainless steel tube; Compression;
16 Hollow ratio; FE modelling.

17 **1. Introduction**

18 Concrete filled double skin steel tube (CFDST) member as presented in Fig. 1 is
19 produced with two concentric steel tubes and concrete filled between two tubes [1-3].
20 In the past twenty years, large numbers of researches have been conducted on the
21 behaviours of such members under different load conditions, including the static [4-9],
22 dynamic [10-12] and fire [13] loadings. Available research results have indicated that
23 the CFDST members present greater flexural capacity and better seismic resistance
24 when compared to the concrete filled steel tube (CFST) members. In addition, owing
25 to the internal steel tube being thermally protected by the concrete, such members also
26 exhibit good fire resistance. Considering above several advantages, this type of
27 composite member has been increasingly utilized in bridge piers, transmission towers
28 and electrical grid structures, etc. [3, 14, 15]

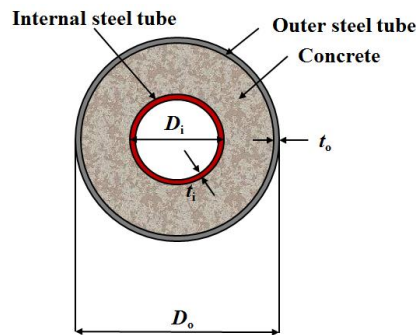


Fig.1. Cross-section of CFDST member.

29 Recently, stainless steel outer tube has been employed in the construction application,
30 which is due to its better corrosion, fire and impact resistances, and maintenance when
31 compared to the carbon steel [16-18]. In addition, stainless steel presents strong strain-
32 hardening behaviour without a definite yield strength and excellent ductility, i.e., the
33 elongation of stainless steel after fracture can reach about 50% [18]. However, the price
34 of stainless steel inhibits its wide application in construction. To economically and
35 efficiently use of this material, a CFDST section with a stainless steel outer tube was
36 developed [19]. Han et al. [19] conducted tests on 80 CFDST stub columns with
37 stainless steel external tube under axial loading, and found that their compression
38 behaviours are similar to those of double carbon-skin composite columns. The ultimate

39 load-bearing capacities of such stub columns were analyzed using finite element
40 methods by Hassanein et al. [20] and Wang et al. [21]. In 2019, Wang et al. [22]
41 experimentally and numerically investigated the compressive behaviours of CFDST
42 short columns having stainless steel external tube and high strength steel internal tube.
43 The results have indicated that the design models for CFST members generally provide
44 conservative predictions of CFDST stub columns. However, limited researches have
45 been conducted on the performance of CFDST slender columns and beams having
46 stainless steel external tube. The only available study on such slender columns was
47 conducted by Hassanein and Kharoob [23] using the FE method. According to
48 numerical results, the authors concluded that the design load-carrying capacities of
49 CFST slender columns given by AISC specification [24] and EC 4 [25] overestimate
50 the compressive capacities of CFDST slender columns with the stainless steel jacket.
51 Until now, no experimental researches on such slender columns and beams have been
52 undertaken.

53 Consequently, this work aims to investigate the performance of the CFDST slender
54 columns and beams with external stainless steel tube. For this purpose, a total of 24
55 specimens were tested. The finite element (FE) models were established to validate the
56 experimental results and employed to perform parametric studies to expand the ranges
57 of hollow ratio, load eccentricity ratio and material strength. The load distribution
58 among the components and contact stress between sandwiched concrete and steel tubes
59 were also investigated using the FE models. Finally, all the FE and experimental results
60 were compared with the load-bearing capacity predictions for the composite members
61 with carbon steel external tube suggested by Han et al. [3] and Chinese GB 50936 [26].

62 **2. Test program**

63 **2.1. Specimen preparations**

64 Twenty-four circular CFDST specimens with internal carbon and external stainless
65 steel tubes were examined, including 18 slender columns and 6 beams. Details about
66 column and beam specimens are presented in Table 1 and Table 2, respectively. The key

67 parameters are hollow ratio, $\chi=D_i/(D_o-2t_o)$ (in which D_i and D_o respectively represent
68 the outer diameter of internal carbon and outer stainless steel tube and t_o is the thickness
69 of the outer tube), slenderness ratio λ and load eccentricity e . The slenderness ratio λ
70 greater than 22 is applied to define the CFDST slender column, as suggested by
71 Hassanein and Kharoob [23]. The identification system of all specimens (Table 1 and
72 Table 2) is defined as follows:

- 73 • The first characters “C” and “B” represent the column and beam specimen,
74 respectively.
- 75 • The first numbers “1”, “2” and “3” stand for the specimen lengths corresponding
76 to 800 mm, 1300 mm and 1800 mm, respectively.
- 77 • The following numbers “0.44”, “0.69” and “0.81” account for the hollow ratio.
- 78 • The next numbers “4” and “14” denote the eccentricity of applied load on the
79 column.
- 80 • The last letters “a” and “b” represent the first and second specimen in one group,
81 respectively.

82 Table 1 Details of column specimens.

No.	Specimen label	External stainless tube	Inner carbon tube	χ	L (mm)	e (mm)	λ
		$D_o \times t_o$ (mm)	$D_i \times t_i$ (mm)				
1	C1-0.44-4-a	114×1.88	48×2.52	0.44	800	4	23
2	C1-0.44-4-b	114×1.88	48×2.52	0.44	800	4	23
3	C1-0.69-14-a	114×1.88	76×2.01	0.69	800	14	22
4	C1-0.69-14-b	114×1.88	76×2.01	0.69	800	14	22
5	C2-0.44-4-a	114×1.88	48×2.52	0.44	1300	4	38
6	C2-0.44-4-b	114×1.88	48×2.52	0.44	1300	4	38
7	C2-0.69-14-a	114×1.88	76×2.01	0.69	1300	14	35
8	C2-0.69-14-b	114×1.88	76×2.01	0.69	1300	14	35
9	C3-0.44-4-a	114×1.88	48×2.52	0.44	1800	4	53
10	C3-0.44-4-b	114×1.88	48×2.52	0.44	1800	4	53
11	C3-0.69-14-a	114×1.88	76×2.01	0.69	1800	14	49
12	C3-0.69-14-b	114×1.88	76×2.01	0.69	1800	14	49

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84
85
86
87

Table 2 Details of beam specimens.

No.	Specimen label	External stainless tube	Inner carbon tube	χ	L	l/D_o
		$D_o \times t_o$ (mm)	$D_i \times t_i$ (mm)			
1	B3-0.44-a	114×1.88	48×2.52	0.44	1800	5
2	B3-0.44-b	114×1.88	48×2.52	0.44	1800	5
3	B3-0.69-a	114×1.88	76×2.01	0.69	1800	5
4	B3-0.69-b	114×1.88	76×2.01	0.69	1800	5
5	B3-0.81-a	114×1.88	89×2.01	0.81	1800	5
6	B3-0.81-b	114×1.88	89×2.01	0.81	1800	5

89

Note: t_i is the internal tube thickness, L is the specimen length, l is the shear span.

90

The specimens were made in the following procedures: both the outer stainless and

91

inner carbon steel tubes were first cut from the long tubes, next, a steel plate was welded

92

to one end of both inner and outer tubes at the designed position. Self-consolidating

93

concrete was then poured into the gap between the inner and outer tubes. For column

94

specimens, the concrete was filled slightly higher than both steel tubes to avoid the gap

95

between the concrete and steel plate. Before testing, the column specimen was surface

96

treated and sealed by the other end plate. For beam specimens, the steel plate was

97

removed and the two ends were uncapped.

98

2.2. Material properties

99

Material properties of carbon and stainless steels were determined from the tensile test

100

in accordance with ISO 6892-1 [27]. All tensile steel coupons were extracted from the

101

steel tubes. Owing to the rounded stress-strain response of stainless steel, the 0.2%

102

proof stress ($\sigma_{0.2}$) is used to specify the yield stress [17, 18]. Table 3 presents the average

103

yield stress σ_y , ultimate stress σ_u , modulus of elasticity E_s and elongation δ for all steels.

104

The cube (150 mm×150 mm×150 mm) compressive strength ($f_{cu,28d}$ and $f_{cu,test}$) and

105

prism (150 mm×150 mm×300 mm) elastic modulus ($E_{c,test}$) are given in Table 4.

106

Table 3 Material properties of carbon and stainless steels.

	σ_y /MPa	σ_u /MPa	E_s /MPa	δ
Carbon steel (2.01 mm)	275	351	2.08×10^5	0.22
Carbon steel (2.52 mm)	276	384	2.05×10^5	0.25
Stainless steel (1.88 mm)	322	703	1.91×10^5	0.46

107

Table 4 Compressive properties of concrete.

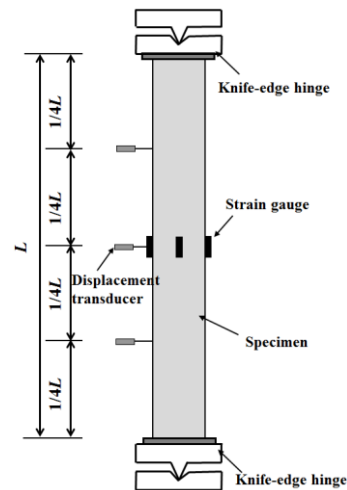
$f_{cu,28d}$ /MPa	$f_{cu,test}$ /MPa	$E_{c,test}$ /MPa
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108 **2.3. Test apparatus and procedures**

109 The column specimens were tested using a hydraulic compression machine with a
 110 loading capacity of 5000 kN. Figs. 2 and 3 present the loading setup and instrument
 111 arrangement for slender columns and beams, respectively. For the slender column under
 112 compression, the high strength steel plate with 6 mm deep groove and the knife edge
 113 were employed at both ends of specimens to achieve the pinned-end conditions and
 114 different loading eccentricities (Fig. 2). The compression load was applied through the
 115 knife edge . Three displacement transducers were respectively employed to monitor the
 116 lateral deflections corresponding to $1/4L$, $1/2L$ and $3/4L$. Strain gauges were used to
 117 monitor the longitudinal strains of stainless steel tube at the $1/2$ -height of the column,
 118 as presented in Fig. 2(b).



(a) Test scene



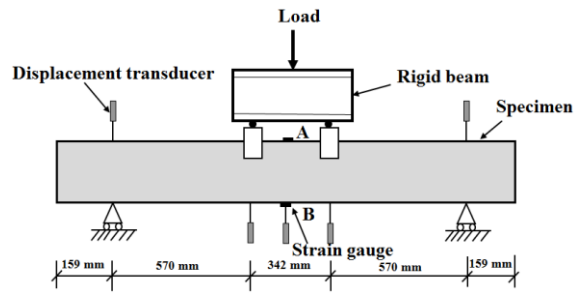
(b) Schematic view

Fig.2. Loading setup and instrument arrangement for slender column specimen.

119 For the prue bending test, a four-points testing rig was employed to apply the moment
 120 as shown in Fig. 3. Han et al. [28] found that the shear span-to-depth ratio varying
 121 between 1.25 and 6 had an insignificant effect on the moment-curvature curves of the
 122 CFST beams. Therefore, the shear span-to-depth ratio of 5 and two-point loads were
 123 used in the test. The in-plane deflections were monitored by three displacement
 124 transducers along the beam. Strain gauges were employed for monitoring the strain
 125 developments at the $1/2$ -span section.



(a) Test scene



(b) Schematic view

Fig.3. Loading setup and instrument arrangement for beam specimen.

126 The load was force-controlled at a rate of 2 kN/s up to approximately 85% of the
 127 estimated load-bearing resistance, and then displacement control was adopted to
 128 capture the post-peak curve of the specimen. The loading interval was less than 10% of
 129 the load-bearing capacity estimated by FE model. Each load interval was sustained for
 130 around 2 min to record the data and observe the phenomenon.

131 2.4. Test results and discussions

132 The failure patterns of all specimens are presented in Fig. 4. For the slender columns, a
 133 typical global buckling with large lateral deflection was observed (Fig. 4(a)). The local
 134 buckling at the 1/2-height section was unobvious due to the presence of sandwiched
 135 concrete, except for specimen C1-0.69-14. Similar failure patterns were also observed
 136 on the conventional CFST and CFDST columns with external carbon steel tube [5, 29-
 137 31]. There was no significant difference in the failure pattern between specimens with
 138 hollow ratios of 0.44 and 0.69.

139 The typical failure pattern of the beam specimens is presented in Fig. 4(b). It is noted
 140 that an outward folding failure formed in the CFDST beams with external stainless steel
 141 tube under bending, all specimens were failed in a ductile manner. This is similar to
 142 that found in the CFST beam [28, 32]. An unobvious difference was observed among
 143 specimens with varying hollow ratio. Unlike the hollow tube, the specimens presented
 144 an insignificant outward local buckling at the compression side. The external steel tube
 145 of beam specimens was removed after bending, as presented in Fig. 4(b). It can be found

146 that the sandwiched concrete maintained intact, and tensile cracks mainly occurred at
 147 the bottom of the 1/2-span section.

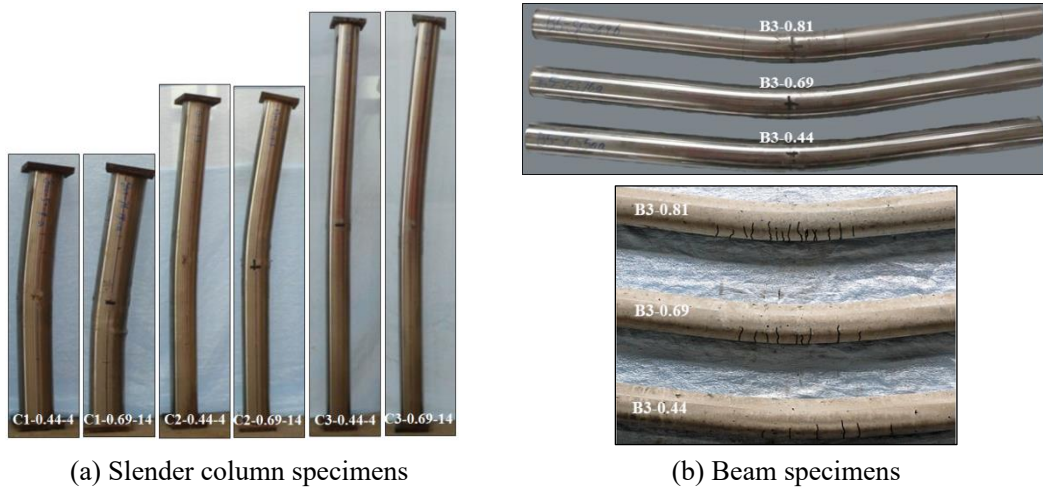


Fig.4. Failure patterns of specimens.

148 The axial load vs. deflection and longitudinal strain at the mid-height of column
 149 specimens are presented in Figs. 5 and 6. Generally, the axial load vs. mid-height
 150 deflection curves exhibited three phases: elastic , elasto-plastic with decreasing
 151 stiffness, and post failure (Fig. 5). The lateral deflection at the mid-height was not
 152 obvious before reaching the maximum load, and it increased rapidly during the post-
 153 peak phase. As expected, the load-carrying capacity of the column specimens decreased
 154 with the increase of slenderness ratio. As presented in Fig. 6, the compression and
 155 tension zones on the mid-height section exhibited simultaneously at the beginning of
 156 eccentric loading stage, mainly due to the obvious second-order effect.

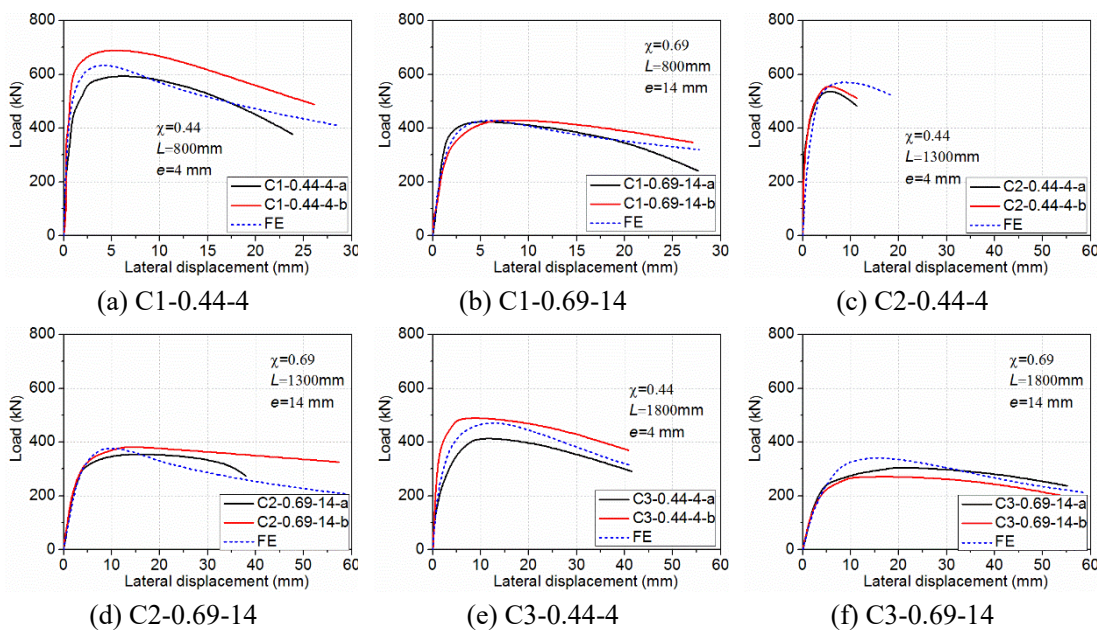


Fig.5. Axial load vs. mid-height deflection (slender column).

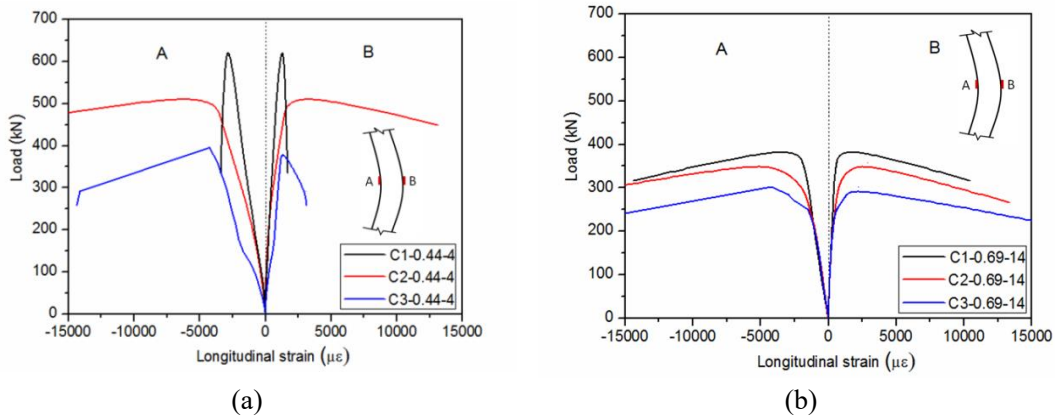


Fig.6. Axial load vs. longitudinal strain at the mid-height (slender column).

157 Figs. 7 and 8 present the moment vs. deflection and longitudinal strain at the mid-
 158 span of beam specimens. It can also be seen from these curves that the CFDST beams
 159 with external stainless steel tube exhibited good ductility. Under pure bending, the
 160 response of all specimens showed elastic and plastic deformation phases until the
 161 ultimate moment resistance was reached. As suggested by Han [32], considering the
 162 practice condition, the moment capacity of composite beam is taken as the moment at
 163 the maximum fiber tensile strain of 10000 $\mu\epsilon$. The moment capacities of beam
 164 specimens with hollow ratios of 0.44, 0.69 and 0.81 were 13.3, 15.0 and 14.0 kN·m,
 165 respectively. Specimens with hollow ratios of 0.69 and 0.81 presented relatively high
 166 values when compared to that having hollow ratio of 0.44, which is mainly owing to
 167 the larger flexural resistance by increasing the diameter of internal steel tube.

168

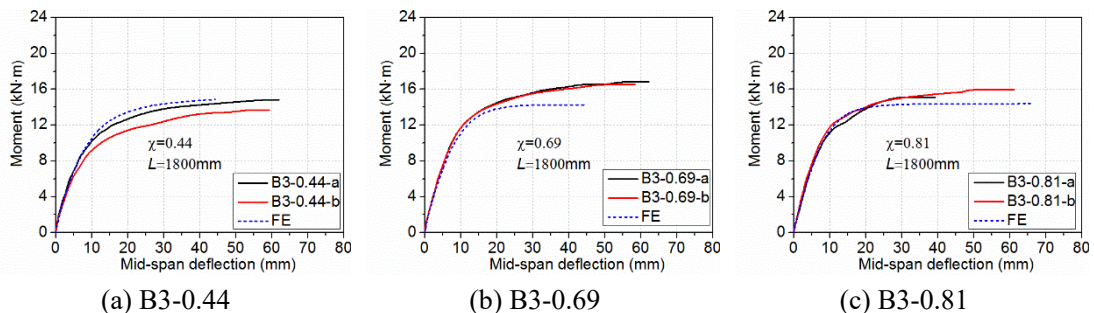


Fig.7. Moment vs. mid-span deflection (beam).

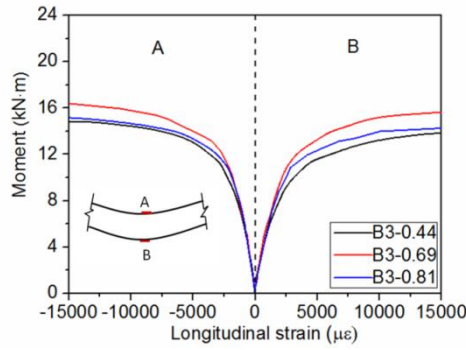


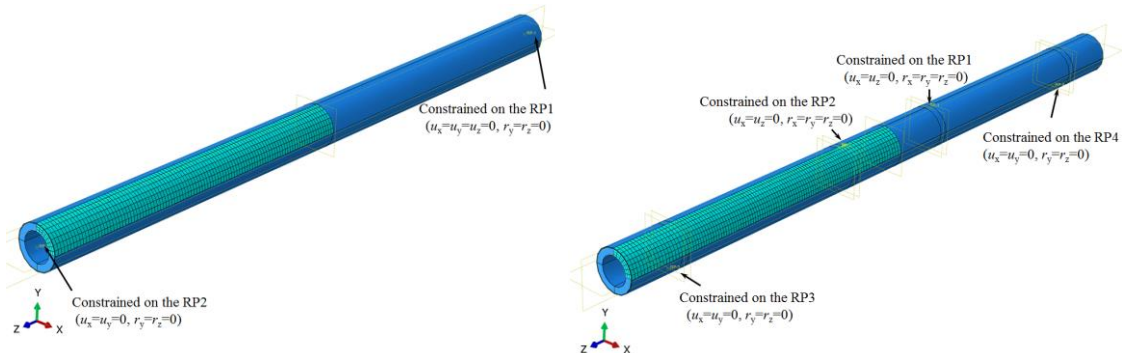
Fig.8. Moment vs. longitudinal strain at the mid-span (beam).

169 **3. Finite element (FE) analysis**

170 **3.1. FE modeling**

171 In this section, to further study the structural behaviours of the CFDST members with
 172 external stainless steel tube under compression and bending, the program ABAQUS
 173 was employed to develop the FE model. The material models and stainless/carbon steel-
 174 concrete interface were presented in detail. These FE models were verified against the
 175 test results.

176 Typical FE models for the slender columns and beams are presented in Fig. 9, where
 177 the boundary condition, loading and mesh size are shown. The reference points in the
 178 models were employed to apply the boundary restraint and loading. C3D8R (8-noded
 179 solid element) and S4R (4-noded shell element) were respectively utilized for the
 180 sandwiched concrete and the carbon/stainless steel tube. It is well documented that the
 181 slender member is affected by the initial global imperfection. There were two phases to
 182 introduce the imperfection. Firstly, the buckling analysis was conducted to obtain the
 183 first buckling mode. In the second phase, the first eigenmode multiplied by a factor of
 184 $0.001L$ (L is the specimen length) was introduced into the loading model [31, 33].



(a) Slender column

(b) Beam

Fig.9.View of FE models for slender column and beam in ABAQUS.

185 In this work, a 5-stage stress-strain model suggested by Han et al. [34] was employed
 186 to model the carbon steel. Stainless steel presents a rounded stress-strain behaviour and
 187 pronounced strain hardening. A 2-stage stress-strain model suggested by Rasmussen
 188 [35] was used to simulate the material stress-strain response of stainless steel, as
 189 presented in Eq. (1).

$$\varepsilon = \begin{cases} \frac{\sigma}{E_0} + 0.002 \left(\frac{\sigma}{\sigma_{0.2}} \right)^n & \sigma \leq \sigma_{0.2} \\ \frac{\sigma - \sigma_{0.2}}{E_{0.2}} + \varepsilon_u \left(\frac{\sigma - \sigma_{0.2}}{\sigma_u - \sigma_{0.2}} \right)^m + \varepsilon_{0.2} & \sigma > \sigma_{0.2} \end{cases} \quad (1a)$$

$$n = \frac{\ln 20}{\ln(\sigma_{0.2}/\sigma_{0.01})} \quad (1b)$$

$$E_{0.2} = \frac{E_0}{1 + 0.002n/e} \quad (1c)$$

$$e = \frac{\sigma_{0.2}}{E_0} \quad (1d)$$

$$m = 1 + 3.5 \frac{\sigma_{0.2}}{\sigma_u} \quad (1e)$$

$$\varepsilon_{0.2} = \frac{\sigma_{0.2}}{E_0} + 0.002 \quad (1f)$$

190 in which σ = stress, ε = strain, E_0 = initial modulus of elasticity of stainless steel, $\sigma_{0.2}$ =
 191 stress corresponding to 0.2% plastic strain, $E_{0.2}$ = tangent modulus at the $\sigma_{0.2}$, σ_u =
 192 ultimate stress, ε_u = ultimate strain, n and m are strain hardening coefficients.

193 Tao et al. [36] found that the confinement effect provided by the stainless steel tube to
 194 the core concrete was similar to that by the carbon steel tube. Therefore, the concrete
 195 compressive model used in the CFDST member with external carbon steel tube [21]
 196 was adopted in the simulation of the sandwiched concrete, as given in Eq. (2).

$$y = \begin{cases} 2x - x^2 & x \leq 1 \\ \frac{x}{\beta_0(x-1)^{\eta+x}} & x > 1 \end{cases} \quad (2a)$$

$$x = \varepsilon / \varepsilon_0 \quad (2b)$$

$$y = \sigma / f_c' \quad (2c)$$

$$\varepsilon_0 = \varepsilon_c + 800 \cdot \xi^{0.2} \cdot 10^{-6} \quad (2d)$$

$$\varepsilon_c = (1300 + 12.5 \cdot f_c') \cdot 10^{-6} \quad (2e)$$

$$\beta_0 = 0.5(2.36 \times 10^{-5})^{[0.25 + (\xi - 0.5)^7]} (f_c')^{0.5} \geq 0.12 \quad (2f)$$

$$\xi = \frac{A_{so} \cdot f_{yo}}{A_{c, \text{nominal}} \cdot f_{ck}} \quad (2g)$$

197 in which $f_c' =$ concrete cylinder strength, $f_{ck} =$ characteristic concrete strength ($f_{ck} = 0.67$
 198 f_{cu} , where $f_{cu} =$ cube strength of concrete), $f_{yo} =$ yield stress of outer steel tube, $A_{so} =$ cross-
 199 section area of the outer tube, $A_{c, \text{nominal}} =$ the nominal cross-section area of the concrete
 200 ($A_{c, \text{nominal}} = \pi (D_o - 2t_o)^2 / 4$); $\eta = 2$ for circular section.

201 The “concrete damaged plasticity model” in ABAQUS material was employed to
 202 describe the inelasticity of the sandwiched concrete. The linear stress-strain model
 203 suggested in Refs [37, 38] was adopted in the modeling of the sandwiched concrete in
 204 tension, as expressed in Eq. (3).

$$\sigma = \begin{cases} E_c \varepsilon & \varepsilon \leq \varepsilon_{cr} \\ f_t' \left(\frac{\varepsilon - \varepsilon_{tu}}{\varepsilon_{cr} - \varepsilon_{tu}} \right) & \varepsilon_{cr} < \varepsilon \leq \varepsilon_{tu} \\ 0 & \varepsilon > \varepsilon_{tu} \end{cases} \quad (3)$$

205 in which $E_c =$ modulus of elasticity of concrete ($E_c = 4700 \sqrt{f_c'}$), $f_t' = 0.1 f_c'$, $\varepsilon_{cr} = f_t' / E_c$,
 206 $\varepsilon_{tu} = 15 \varepsilon_{cr}$.

207 The interaction behaviour between the sandwiched concrete and steel tube was defined
 208 by the Coulomb’s friction model along the tangential direction and hard-contact model
 209 along the normal direction. The sandwiched concrete was chosen as the master surface,
 210 where the surface of external/internal steel tube was defined as the slave. The friction
 211 coefficient between the concrete and the stainless steel tube was adopted as 0.25 [23],
 212 while the value of 0.6 was defined between the concrete and carbon steel tube. The
 213 appropriate mesh density was determined by the mesh convergence studies. A mesh
 214 size of $D_o/20$ over the cross-section was chosen for the model, where D_o is the the outer
 215 diameter of external steel tube.

216 3.2. Verification of the FE model

217 In order to verify the FE models, the predicted curves of load vs. mid-span deflection
 218 are compared with those obtained experimentally, as presented in Figs. 5 and 7. The
 219 features of the complete test curve include the stiffness, ultimate strength and load-
 220 deflection development of the specimen. Fig. 10 shows the comparisons between

221 experimental and numerical ultimate loads. The average ratio of the FE results to
 222 experimental results is 0.97, with a standard deviation of 0.01. Generally, the FE models
 223 could replicate the load-deflection curves and failure patterns for the tested CFDST
 224 slender columns and beams having stainless steel external tube. In some cases, the
 225 predicted curves are not the same with the experimental results, mainly owing to the
 226 experimental error and the material property deviations between the simulation and test.

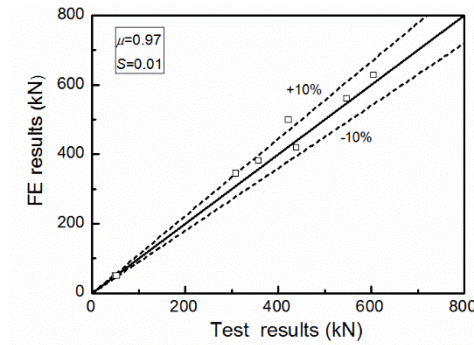


Fig.10. Comparison between FE and test results.

227 4. Parametric investigations and design methods

228 4.1. Parametric investigations

229 After validating the FE methodmodels, extensive parametric investigations were
 230 conducted to extend the ranges of hollow ratio, load eccentricity ratio, yield stress of
 231 external and internal steel tubes and concrete strength. The parameters investigated are
 232 presented in Tables 5 and 6. The yield strength of stainless steel is taken as the stress at
 233 0.2% plastic strain. In order to investigate the hollow ratio (χ) on the behaviour of
 234 slender columns and beams, the only dimension variation was made in the internal tube
 235 diameter (D_i).

236 **Table 5** Parametric investigations for slender columns.

Case	Outer tube		Inner tube		χ	L (mm)	λ	e/r_o		f_{yo} (MPa)	f_{yi} (MPa)	f_{cu} (MPa)
	D_o (mm)	t_o (mm)	D_i (mm)	t_i (mm)								
Column	400	10	114	4	0.3	4500	43.0	0,	0.2,	230,	235,	30, 50
								0.4		380	390	
	400	10	190	4	0.5	4500	41.0	0,	0.2,	230,	235,	30, 50
								0.4		380	390	
	400	10	266	4	0.7	4500	37.0	0,	0.2,	230,	235,	30, 50
								0.4		380	390	
	600	10	174	4	0.3	4500	29.0	0,	0.2,	230,	235,	30, 50
								0.4		380	390	
	600	10	290	4	0.5	4500	27.0	0,	0.2,	230,	235,	30, 50
								0.4		380	390	

600 10 406 4 0.7 4500 25.0 0, 0.2, 230, 235, 30, 50
0.4 380 390

237

Table 6 Parametric investigations for beams.

Case	Outer tube		Inner tube		χ	L (mm)	l/D_o	f_{y_o} (MPa)	f_{y_i} (MPa)	f_{c_u} (MPa)
	D_o (mm)	t_o (mm)	D_i (mm)	t_i (mm)						
Beam	400	10	114	4	0.3	4500	5	230, 380	235, 390	30, 50
	400	10	190	4	0.5	4500	5	230, 380	235, 390	30, 50
	400	10	266	4	0.7	4500	5	230, 380	235, 390	30, 50
	600	10	174	4	0.3	4500	5	230, 380	235, 390	30, 50
	600	10	290	4	0.5	4500	5	230, 380	235, 390	30, 50
	600	10	406	4	0.7	4500	5	230, 380	235, 390	30, 50

238

Note: L is the column and beam length; f_{y_o} and f_{y_i} are respectively the yield strength of outer and

239

inner steel tube; f_{c_u} is the cubic compressive strength of sandwiched concrete; e is the load

240

eccentricity; r_o is the outer radius of external steel tube; l is the shear span.

241

4.1.1 Influences of key parameters

242

Fig. 11 presents the effects of key parameters on the load-carrying capacities of slender

243

columns. As shown in Fig. 11(a), for columns with a diameter of 600 mm, increasing

244

hollow ratio from 0.3 to 0.5 hardly affects the ultimate strengths. Columns having

245

hollow ratio of 0.7 show the lowest load-carrying capacities. For columns with a

246

diameter of 400 mm, the highest ultimate strength was found at the hollow ratio of 0.5.

247

Above changes are mainly related to the variations in the cross-sectional area of

248

concrete and the flexural rigidity of the inner steel tube. The reduction in the area of

249

concrete induces a decrease in the ultimate strength, whereas the increasing flexural

250

rigidity caused by moving the inner tube farther from the centroid increases the column

251

strength. In Fig. 11, the load-carrying capacities decrease with the increase in the load

252

eccentricity ratio, and increase with the increasing yield stress of outer steel tube and

253

concrete strength. It can be seen that the strength of internal steel tube has a minor effect

254

on the ultimate load of slender columns. This is mainly because that the flexural

255

resistance is marginally affected by the variation in the strength of inner steel tube

256

which is located near the neutral axis.

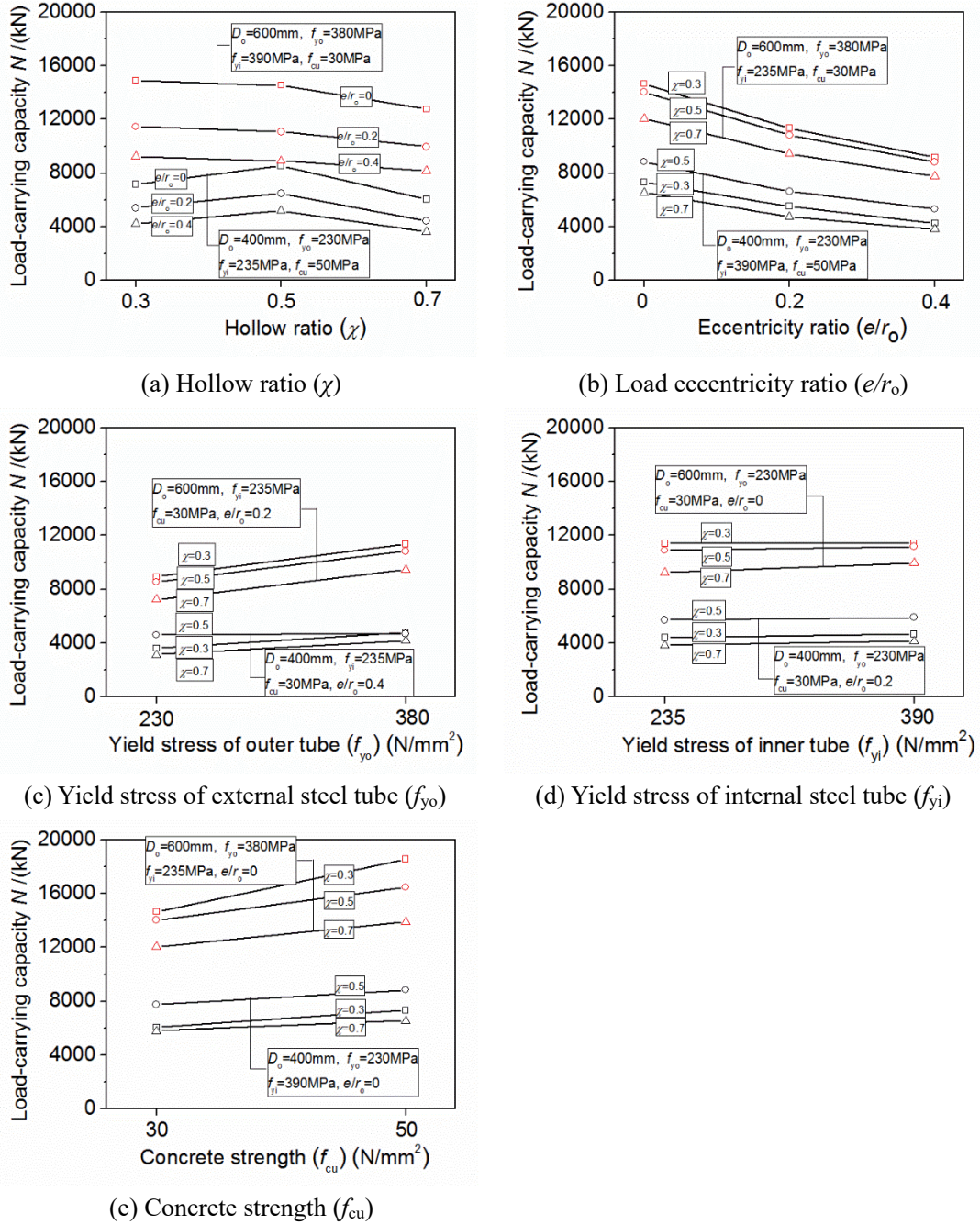


Fig.11. Effect of parameters on the load-carrying capacities of slender columns.

257 The effects of these parameters on the moment capacities of beams are presented in Fig.
 258 12. Similar to the load-carrying capacities of slender columns, the bending capacities
 259 of beams also increase significantly with increasing yield stress of external steel tube.
 260 As with the hollow ratio increases from 0.3 to 0.7, the change in the bending capacity
 261 is not obvious. The strengths of internal steel tube and concrete have a marginal effect
 262 on the flexural resistance. Based on the analysis of Figs. 11 and 12, it is structural
 263 efficiency to employ the high-strength outer steel tube and hollow ratio of 0.5 in CFDST

264 member under combined compression and bending.

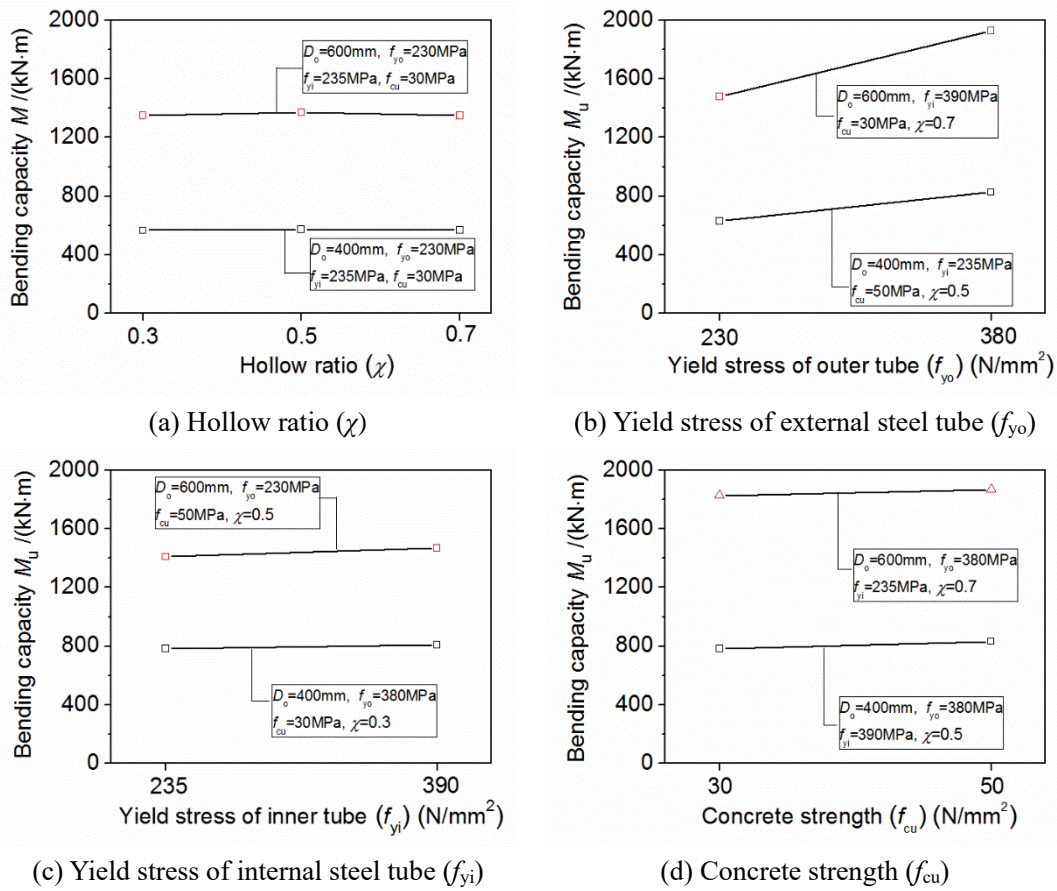


Fig.12. Effect of parameters on the moment capacities of beams.

265 4.1.2 Load distribution and confinement effect

266 The load distributions among the outer and inner steel tubes as well as sandwiched
 267 concrete were analyzed, as shown in Fig. 13. For the slender column, all the steel tubes
 268 and concrete components carry the compression load during the whole loading phase,
 269 and sandwiched concrete contributes a large portion of the axial resistance (Fig. 13(a)).
 270 At the ultimate strength, the axial-load contribution percentages of sandwiched
 271 concrete, outer and inner steel tubes are 58.1%, 30.9% and 11.0%, respectively. Due to
 272 the confinement from the steel tube, the load carried by the whole CFDST member
 273 declines, while the load of the sandwiched concrete remains increasing. For the beam
 274 in Fig. 13(b), the majority of bending moment is carried by the outer steel tube because
 275 of the large distance from the neutral axis.

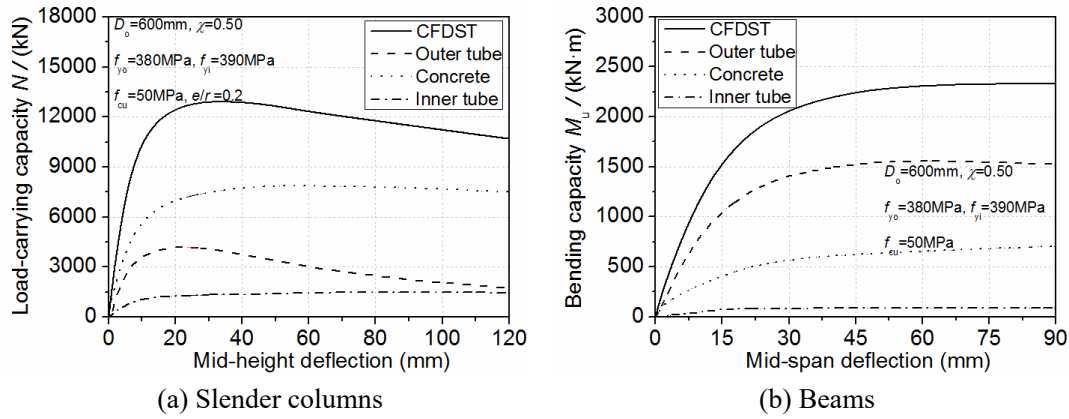


Fig.13. Load distributions in slender columns and beams.

276 Fig. 14 presents the contact stress between steel tube and sandwiched concrete at the
 277 mid-span section of slender columns and beams. For CFDST slender column in Fig.
 278 14(a), the contact stress does not exist in the initial phase of loading because the
 279 Poisson's ratio of sandwiched concrete is smaller compared with that of the steel tube.
 280 With the increasing load, the contact stress between the outer tube and concrete begins
 281 to develop, which is mainly due to enlarged lateral expansion of the concrete exceeds
 282 that of the steel tube during the elastic-plastic phase. It clearly shows that the
 283 confinement provided by the outer tube is greater than that by the inner tube in Fig.
 284 14(a). Due to the large deformation of the inner steel after the steel comes into the
 285 plastic stage, the pressure between the inner steel tube and sandwiched concrete
 286 increases in the late phase of loading. For CFDST beam, presented in Fig. 14(b), the
 287 maximum contact stress occurs at the tensile side (point 4) between the outer steel tube
 288 and sandwiched concrete. This is because the concrete at point 4 undergoes the largest
 289 deformation in the section. Therefore, the outer steel tube exerts stronger contact stress
 290 to the sandwiched concrete.

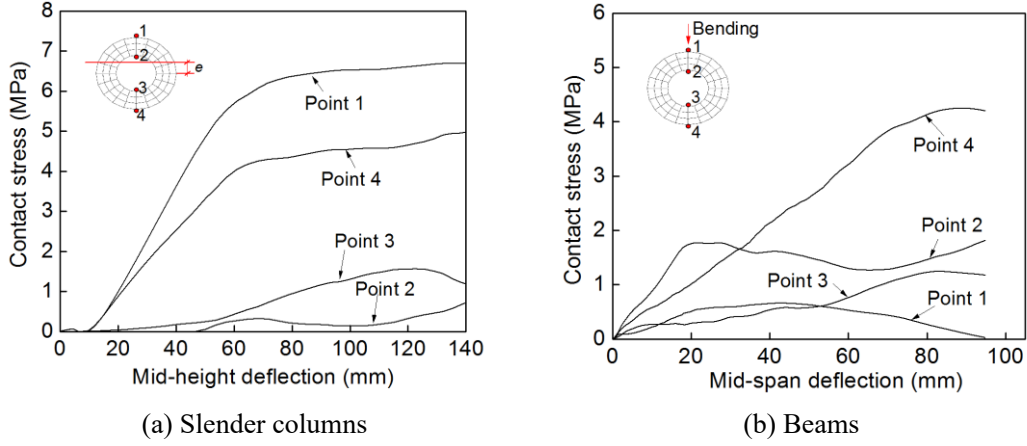


Fig.14. Contact stress in the slender columns and beams.

291 4.2. Comparison with current design methods

292 Until now, there is no available design method for the CFDST slender columns and
 293 beams with stainless steel external tube. In 2018, Han et al. [3] proposed the design
 294 methods for estimating the load-bearing capacities of CFDST members with external
 295 carbon steel tube. Previous study conducted by Han et al. [19] showed that the formula
 296 to calculate the ultimate strength of the carbon steel CFDST stub column can be applied
 297 for such stub column with stainless steel external tube. Thus, in this section, this design
 298 rule suggested by Han et al. [3] is compared with the experimental and FE results to
 299 assess their applicability in CFDST slender columns and beams with external stainless
 300 jacket. In addition, the method for the carbon steel hollow CFST members in GB 50936
 301 [26] is also modified by considering the contribution of the inner steel tube to predict
 302 the load-carrying capacities of CFDST members.

303 4.2.1 Design method by Han et al.

304 According to Han et al. [3], the buckling capacity N of the CFDST column is as follows:

$$N = \varphi N_u \quad (4a)$$

where

$$\varphi = \begin{cases} 1 & \lambda \leq \lambda_o \\ a \cdot \lambda^2 + b \cdot \lambda + c & \lambda_o < \lambda \leq \lambda_p \\ \frac{d \cdot (-0.23\chi^2 + 1)}{(\lambda + 35)^2} & \lambda > \lambda_p \end{cases} \quad (4b)$$

$$N_u = f_{osc} (A_{so} + A_c) + f_{yi} A_{si} \quad (4c)$$

$$f_{osc} = \alpha / (1 + \alpha) \cdot \chi^2 \cdot f_{yo} + (1 + \alpha_n) / (1 + \alpha) \cdot (1.14 + 1.02\xi) \cdot f_c \quad (4d)$$

$$\zeta = \frac{A_{so} \cdot f_{yo}}{A_{c, \text{nominal}} \cdot f_c} = a_n \cdot \frac{f_{yo}}{f_c} \quad (4e)$$

$$a = A_{so}/A_c \quad (4f)$$

305 in which N_u = sectional capacity under compression, φ = buckling reduction coefficient,
 306 f_{osc} = compound compressive strength of the concrete and the outer steel tube, a , b , c
 307 and d are the parameters related to the buckling reduction coefficient, as presented
 308 detailly in Ref. [5].

309 The flexural capacity M_u of the CFDST member is expressed as:

$$M_u = \gamma_{m1} \cdot W_{scm} \cdot f_{osc} + \gamma_{m2} \cdot W_{si} \cdot f_{yi} \quad (5a)$$

where

$$W_{scm} = \frac{\pi(D_o^4 - D_i^4)}{32D_o} \quad (5b)$$

$$W_{si} = \frac{\pi[D_i^4 - (D_i - 2t_i)^4]}{32D_i} \quad (5c)$$

$$\gamma_{m1} = 0.48 \ln(\zeta + 0.1) \cdot (1 + 0.06\chi - 0.85\chi^2) + 1.1 \quad (5d)$$

$$\gamma_{m2} = -0.02\chi^{-2.76} \ln \zeta + 1.04\chi^{-0.67} \quad (5e)$$

310 in which W_{scm} = compound section modulus of the concrete and the outer steel tube,
 311 W_{si} = section modulus of the internal steel tube.

312 The axial load N vs. bending moment M relationship of the CFDST member under
 313 combined compression and bending is presented in Eq. (6):

$$\begin{cases} \frac{N}{\varphi N_u} + \frac{a_1}{d_1} \cdot \left(\frac{\beta_m \cdot M}{M_u} \right) = 1 & \text{for } NN_u \geq 2\varphi^3 \cdot \eta_o \\ -b_1 \cdot \left(\frac{N}{N_u} \right)^2 - c_1 \cdot \left(\frac{N}{N_u} \right) + \frac{1}{d_1} \cdot \left(\frac{\beta_m \cdot M}{M_u} \right) = 1 & \text{for } NN_u < 2\varphi^3 \cdot \eta_o \end{cases} \quad (6)$$

314 in which β_m = equivalent moment coefficient, as given in EC 4 (Table 6.4) [25], a_1 , b_1 ,
 315 c_1 and d_1 are the parameters to control the N - M relation.

316 4.2.2 GB 50936

317 A unified equation is developed in Chinese code GB 50936 [26] for both solid and
 318 hollow CFST members, considering the influence of hollow ratio χ on the ultimate
 319 strengths. In this part, the cross-sectional compressive and flexural resistances in GB
 320 50936 are modified by adding the contribution of the inner steel tube, as presented in
 321 Eqs. 7 and 8. The modified GB 50936 equations for the axial load-carrying capacity of

322 slender column is expressed as:

$$N = \varphi \left[(A_{so} + A_c) f_{osc} + A_{si} f_{yi} \right] \quad (7a)$$

where

$$\varphi = \frac{1}{2\bar{\lambda}_{osc}^2} \left[\bar{\lambda}_{osc}^2 + (1 + 0.25\bar{\lambda}_{osc}) - \sqrt{\left(\bar{\lambda}_{osc}^2 + (1 + 0.25\bar{\lambda}_{osc}) \right)^2 - 4\bar{\lambda}_{osc}^2} \right] \quad (7b)$$

$$\bar{\lambda}_{osc} = (\lambda_{osc} / \pi) \sqrt{f_{osc} / E_{osc}} \quad (7c)$$

$$f_{osc} = (1.212 + B\zeta + C\zeta^2) \cdot f_c \quad (7d)$$

$$E_{osc} = (E_{so} I_{so} + E_{si} I_{si} + E_c I_c) / I_{osc} \quad (7e)$$

$$\zeta = \frac{A_{so} \cdot f_{yo}}{A_c \cdot f_c} = \alpha \cdot \frac{f_{yo}}{f_c} \quad (7f)$$

323 in which $\bar{\lambda}_{osc}$ = non-dimensional slenderness; E_{osc} = Composite bending modulus.

324 The flexural capacity of the CFDST beam is given by

$$M_u = \gamma_m (W_{scm} \cdot f_{osc} + W_{si} \cdot f_{yi}) \quad (8a)$$

where

$$\gamma_m = (1 - 0.5\chi)(-0.483\zeta + 1.926\sqrt{\zeta}) \quad (8b)$$

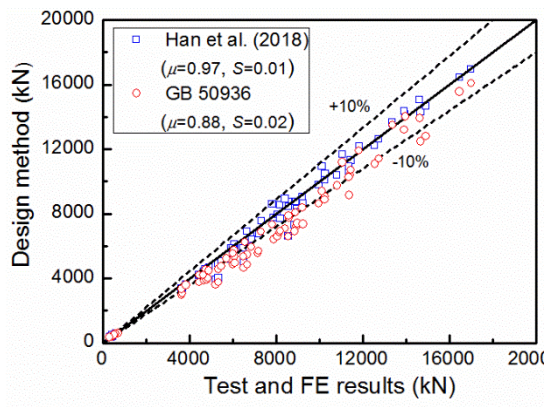
325 The load-carrying capacity of the CFDST beam-column is given as

$$\begin{cases} \frac{N}{\varphi N_u} + \frac{\beta_m \cdot M}{1.5 M_u (1 - 0.4N/N_E)} = 1 & \text{for } NN_u \geq 0.255 \\ -\frac{N}{2.17 N_u} - \frac{\beta_m \cdot M}{M_u (1 - 0.4N/N_E)} = 1 & \text{for } NN_u < 0.255 \end{cases} \quad (9a)$$

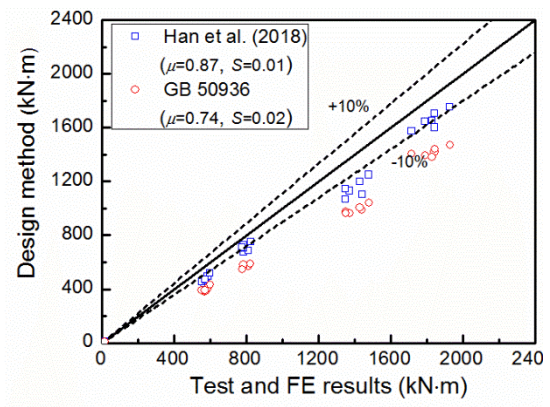
where

$$N_E = \frac{\pi^2 E_{osc} (A_{so} + A_{si} + A_c)}{\lambda_{osc}^2} \quad (9b)$$

326 The predicted ultimate compressive loads of slender columns and moment capacities
 327 of beams using Eqs. (4-9) are compared with the test and FE results in Fig. 15. The
 328 average ratio μ of the predicted results to experimental and FE results and
 329 corresponding standard deviation S are also given in Fig. 15. The comparison results
 330 demonstrate that generally, the design methods recommended by Han et al. [3] are
 331 acceptable for the design of the CFDST slender columns and beams having either
 332 external stainless or carbon steel tube. In general, the GB 50936 provides the
 333 conservative prediction for the ultimate strengths and bending capacities.



(a) Ultimate strengths of slender columns



(b) Moment capacities of beams

Fig.15. Comparison between design method and test and FE results.

334

335 **5. Conclusions**

336 This work experimentally and numerically investigated the behaviours of CFDST
337 slender columns and beams with stainless steel external tube. A total of 24 specimens
338 were tested to obtain their failure patterns and load-deflection curves. The FE models
339 were established to predict the experimental results and used to extend the parameter
340 ranges. The obtained experimental and FE results were used to evaluate the
341 acceptability of the design methods for CFDST members with carbon steel external
342 tube proposed by Han et al. [3]. Within the parameter ranges of this work, the main
343 conclusions are summarized as follows:

- 344 (1) The tested slender columns and beams presented ductile behaviour. The sandwiched
345 concrete remained intact, which is mainly due to the confinement of the double-
346 skin tubes. The failure patterns of CFDST members with external stainless steel
347 tubes were similar to those of the CFST and CFDST members with outer carbon
348 steel.
- 349 (2) The predicted load-bearing capacities and load-deflection developments of slender
350 columns and beams using the finite element (FE) model present reasonable
351 agreements with the experimental results. Through the parametric investigations,
352 considering the structural efficiency, it is advised to adopt the high-strength outer
353 steel tube and hollow ratio of 0.5 in CFDST member under combined compression
354 and bending.
- 355 (3) The analysis of load distribution and confinement effect indicate that during the
356 whole loading process, the external stainless steel tube, sandwiched concrete and
357 inner carbon steel tube in the CFDST slender columns and beams could work
358 together.
- 359 (4) The design methods for estimating the load-carrying capacities of CFDST slender
360 columns and beams with external carbon steel tube proposed by Han et al. [3] yield
361 acceptable predictions for such kind members with external stainless steel tube.

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