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Axial compressive behaviour of CFDST stub columns with large void ratio

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Abstract: This paper describes the experimental and numerical study on the axial compressive behaviour of circular-in-circular concrete-filled double-skin steel tube (CFDST) stub columns with large void ratio. Nine specimens with various void ratio (R_v), diameter-to-thickness ratio of outer tube (D_o/t_o) and compressive strength of concrete (f_{cu}) were tested under axial compression. The failure patterns, load versus displacement (strain) relationship, ultimate capacity and elastic composite stiffness of the specimens were investigated. The experimental results show that all specimens have good structural performance. The typical failure patterns of this new type of columns include local buckling of outer or inner tubes and crushing of concrete infill at the primary buckling locations of both tubes. It is observed from the tests that, there are three key stages in the load versus displacement (strain) relationship, namely: approximative elastic, elastoplastic and nonlinear post-peak, and with the augment of R_v and D_o/t_o and the reduce of f_{cu} , the ultimate capacity and elastic composite stiffness of the specimens decrease. Apart from tests, a finite element (FE) model was developed to further study the axial compressive behaviour of circular-in-circular CFDST stub columns with large void ratio, and the model was validated against the experimental results. Finally, a simplified analytical model to predict the ultimate capacity of circular-in-circular CFDST stub columns with large void ratio was developed, and the accuracy of the model was verified by the available experimental results. It can be used by the practising engineers in the future design of this type of columns.

Keywords: circular-in-circular CFDST stub columns; large void ratio; axial compressive behaviour; experiments; finite element (FE) model; ultimate capacity prediction.

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33 **1. Introduction**

34 Concrete-filled double-skin steel tube (CFDST), which is composed of two concentric steel tubes
35 with different outer perimeter and concrete infill between the two tubes, is a new type of composite
36 structure [1]. Generally, while the ultimate capacity kept constant, CFDST has wider cross-section
37 and greater flexural stiffness than the conventional concrete-filled steel tube (CFST), as the inner steel
38 tube replaces part of the concrete core for this new type of sections. Moreover, CFDST generally uses
39 less materials, and therefore has lower self-weight, better fire resistance and superior seismic
40 performance compared to the conventional CFST [2-4], especially when a higher flexural stiffness is
41 required, [such as viaduct piers, mega columns in various bridges](#), space structures and high-rise
42 buildings, and so on [5, 6]. Furthermore, the CFDSTs consist of inner and/or outer stainless steel
43 tubes with a larger internal space can be used as offshore platform legs, submarine pipelines,
44 underground integrated pipeline corridors and cabins, etc. [7], as they can provide higher axial
45 capacity and stiffness as well as better durability compared to the traditional steel or reinforced
46 concrete. Nowadays, circular-in-circular CFDSTs have been employed in part of electricity poles and
47 high-rise bridge piers [8, 9], in which the void ratio of the CFDST members (R_v) is determined in
48 accordance to design guidelines [4], and R_v is defined as $D_i/(D_o - 2t_o)$, where D_i and D_o are
49 the outer diameter of inner and outer tube respectively, and t_o is the thickness of outer tube.

50 In order to well assist the engineering practice, researchers have conducted comprehensive studies
51 on the structural behaviour of CFDST members, joints and frames while subjected to different loading
52 cases, such as short-term static loading [5, 6, 10-12], long-term sustained loads [13], axial and lateral
53 partial compression [14, 15], dynamic loading [16-18], fire exposure [2, 19, 20], etc. Moreover,
54 specification or guidelines [4, 21] for the design of CFDST structures have also been issued.

55 However, previous studies and available design specification or guidelines focus more on the
56 CFDST members with a relatively small R_v (i.e. $R_v \leq 0.75$). A review of the literature indicates
57 that, there are only a few experimental researches related to circular-in-circular CFDST stub columns
58 with R_v larger than or equal to 0.8, including 10 specimens using polymer concrete with R_v of 0.8

59 to 0.88 [22], 2 specimens using normal concrete with R_v of 0.8 [23], 2 specimens using grout with
60 R_v of 0.87 [24], and 4 specimens using grout with R_v of 0.84 to 0.92 [25], and the variation of R_v
61 with D_o of the existing specimens is displayed in Fig. 1(a). It is shown that, D_o of the specimens
62 with $R_v \geq 0.8$ is generally smaller than 200 mm, and only Li et al. [25] reported two specimens with
63 D_o of 450 mm and R_v of 0.92; however, the infill between two tubes was the grout. In addition, the
64 variation of capacity index (CI) with D_o is demonstrated in Fig. 1(b), and CI is defined as
65 $N_{ue}/(f_{yo} \cdot A_{so} + f_{yi} \cdot A_{si} + f'_c \cdot A_c)$, where N_{ue} is the experimental ultimate capacity, f_{yo} and f_{yi}
66 are the yield strength of outer and inner tube respectively, A_{so} and A_{si} are the area of outer and
67 inner tube respectively, f'_c is the cylindrical compressive strength of concrete, and A_c is the area of
68 concrete. The results in Fig. 1(b) demonstrate that, CI has no consistent variation tendency with the
69 increase of D_o , which means that there is no obvious size effect within D_o range of the existing
70 specimens. Most recently, Wang et al. [7] carried out the finite element (FE) modelling of the CFDST
71 stub columns consisting of outer stainless and inner carbon steel tubes, and the maximum void ratio
72 and outside diameter of the models reached to 0.9 and 220 mm, respectively.

73 Therefore, further studies are needed to evaluate the structural performance of CFDST members
74 with large void ratio beyond the current limit stipulated in the design specification or guidelines and
75 to propose a new limit on the void ratio. An investigation into axial compressive behaviour of circular-
76 in-circular CFDST stub columns with large void ratio is presented in this paper. The influence of void
77 ratio (R_v), diameter-to-thickness ratio of outer tube (D_o/t_o) and compressive strength of concrete
78 (f_{cu}) on the static performance of axially compressed CFDST stub column specimens was
79 experimentally studied. A nonlinear FE model was also constructed using ABAQUS [26] to simulate
80 the responses of circular-in-circular CFDST stub columns with large void ratio while subjected to
81 axial compression, with its accuracy verified by the experimental results. Eventually, a simplified
82 analytical model for predicting the ultimate capacity of circular-in-circular CFDST stub columns with
83 large void ratio was developed.

84

85 **2. Experimental investigation**

86 **2.1 Specimens**

87 Nine circular-in-circular CFDST stub column specimens, consisting of outer and inner steel tubes of
88 circular hollow section (CHS) and concrete infill between them, were tested. Fig. 2 illustrates the
89 cross-section of the specimens, where t_i is the thickness of inner steel CHS. All specimens had the
90 identical height (H) of 1500 mm. The outside diameter of the specimens was 538 mm to achieve a
91 large void ratio, which represents the actual section size of the CFDST members in some practical
92 construction projects. The design parameters of the specimens are presented in Table 1, where f_{cu} is
93 the cubic compressive strength of concrete while conducting the axial compressive tests of the
94 CFDST stub columns, K_e is the elastic compressive stiffness of the specimens, and N_{ufe} is the
95 simulated ultimate capacity based on the FE method described below.

96 The experiment was performed to investigate the influence of R_v (from 0.8 to 0.9), D_o/t_o (95.6
97 and 143.1) and f_{cu} (66.6 MPa and 49.4 MPa) on the behaviour of axially compressed circular-in-
98 circular CFDST stub columns with large void ratio.

99 Outer and inner CHSs were all manufactured by rolling a mild steel sheet of fixed sizes and welding
100 with one straight butt weld, and the circumferential difference between the weld of the inner and outer
101 steel CHSs was about 90 degrees (see Fig. 2). The welding was conducted by using the electrodes
102 having nominal yield stress, ultimate stress and elongation of 330 MPa, 415 MPa and 22%,
103 respectively. The quality of welding was controlled carefully to ensure effective force transmission.
104 To facilitate the casting of concrete, one circular endplate with diameter slightly larger than D_o and
105 thickness of 15 mm was welded to one end of the outer and inner steel CHSs simultaneously. After
106 curing the concrete for 14 days, the top surface of the concrete infill was polished to level with both
107 steel CHSs to ensure that both steel CHSs and the concrete infill could bear the external loads together.
108 In addition, to prevent damage at the loading end, therefore to control the loading process effectively,
109 two circular steel sleeves with adjustable diameters and height of 100 mm were used for restraining
110 the local deformation of the specimen ends.

111 **2.2 Material properties**

112 Carbon steel sheets of different thickness were used for fabricating the CHSs. The properties of steel
113 were acquired by the standard tension coupon tests performed on three specimens, and the measured
114 average values of thickness, yield strength (f_y), tensile strength (f_u), modulus of elasticity (E_s),
115 Poisson's ratio (μ_s) and elongation after fracture (δ_{ef}) are presented in Table 2.

116 Two concrete mixes were produced to fill into the gap between the inner and outer steel CHSs, and
117 the mix proportions of concrete are given in Table 3. Portland cement of 42.5 grade, limestone gravel
118 having particle size of 5-10 mm, river sand, running water and high range water-reducing admixture
119 (Standard Type) were used in producing the concrete. The properties of concrete are presented in
120 Table 3, where $f_{cu,28}$ and f_{cu} are the cubic compressive strength at 28-day and while the tests of
121 CFDST stub column specimens conducted.

122 **2.3 Test set-up and instrumentation**

123 Fig. 3 demonstrates the test set-up for circular-in-circular CFDST stub column specimens under axial
124 compression, and the tests were carried out by a 10,000 kN capacity tester. To monitor the strains and
125 axial displacements, on the outside surface of the outer steel CHS, two cross-sections (S1 and S2)
126 with 4 measuring points along the circumference with 90 degrees apart were chosen to paste the axial
127 and hoop strain gauges (SGs), and there were four displacement transducers (DT) on the lower platen
128 of the tester, as shown in Fig. 3. One load cell was used to gather the varied compressive loads. It
129 should be noted that, a rigid plate consisting of four 20 mm thick steel plates welded together was
130 placed between the upper platen of the tester and the top endplate of the steel sleeve to ensure uniform
131 loading upon the cross-section, considering that the outside diameter of the load cell is smaller than
132 the inner diameter of the inner steel CHS of the specimens.

133 The specimens were continuously loaded until the tests were terminated, and the history of loads
134 and deformations (axial displacements and strains) and the failure process of the specimens were
135 recorded in time. Displacement control method was adopted in this study, i.e., the displacement rate
136 equaled to 0.2 mm/min before reaching the peak load, whilst it was equal to 1.0 mm/min after the

137 peak load was achieved. The tests were terminated when the axial displacement reached one fortieth
138 of the height, or in the post-peak stage the load on the specimen was less than 60% of the measured
139 peak load.

140 **2.4 Test results and discussion**

141 Generally, with the increase of the axial displacements, the loads undertaken by the specimens
142 experienced three key stages of change, i.e., approximately linear increment in the range of 50~60%
143 of the peak load, subsequent nonlinear increment until the peak load and nonlinear reduction after
144 achieving the peak load, and all specimens had good load-carrying capacity and deformability.
145 Simultaneously, when the load reached 80~90% of the peak load, the outer tube began to buckle
146 locally in a number of positions together with the sound of concrete crushing, and in the post-peak
147 phase the buckling range and deformation of the outer tube grew rapidly while increasing the axial
148 displacements.

149 The final failure pattern of the specimens (see Fig. 4) was manifested as outward local buckling
150 (i.e. elephant's foot buckling) of the outer steel CHS (indicated by arrows), and slant of the upper
151 endplate as there is a spherical hinge on the upper platen of the tester. For the specimens having
152 $D_o/t_o = 143.1$ (Groups Aa and Ab), R_v and f_{cu} generally have no obvious impact on the mode
153 and position of local buckling of outer steel CHS, i.e. elephant's foot buckling of outer steel CHS is
154 only observed at the region near two sleeves (see Figs. 4(a) and (b)), considering that the stress state
155 of outer tube near the sleeve is changed owing to additional lateral restraints from the sleeve, and
156 their D_o/t_o value is higher than the limit for the diameter-to-thickness ratio of the circular-in-
157 circular CFDST (i.e. $135(235/f_{y0})$) specified in Han et al. [4], which leads to more local buckling
158 trend and weaker confinement to the concrete infill, and thus the outer tube cannot continue to transfer
159 axial loads when local buckling occurs. For the specimens with $D_o/t_o = 95.6$ (Group Bb),
160 elephant's foot buckling of outer steel CHS appears not only at the region near one sleeve, but also at
161 the region near the mid-height of specimens having $R_v=0.85$ and 0.9 (see Fig. 4(c)), and the local
162 buckling of outer steel CHS at the region near the mid-height of specimen with R_v of 0.9 is the most

163 significant, i.e. R_v has an effect on the local buckling position of outer steel CHS of certain group
164 specimens. On the one hand, the outer steel CHS with D_o/t_o lower than the specified limit has a
165 better local stability and a stronger confinement to the concrete infill, and local buckling at the region
166 near one sleeve cannot stop the transmission of axial loads, which may produce new local buckling
167 at a different position, while on the other hand, with the increase of R_v , D_i/t_i of cross-section of
168 inner tube increases and the concrete area decreases, which leads to the weakening of the local
169 stability of inner tube and the supporting effect of concrete on the local buckling of both tubes. In
170 addition, while keeping other parameters the same, the specimen with a smaller D_o/t_o has a smaller
171 outward local buckling deformation of outer steel CHS, as shown in Figs. 4(b) and (c), considering
172 that the concrete infill is better confined by the outer tube with a smaller D_o/t_o [23]. Generally, R_v
173 and f_{cu} have no consistent effect on the local buckling deformation of outer steel CHS.

174 It was found that, the inner steel CHS had inward and/or outward local buckling along the height
175 direction within the buckling range of outer tube, which indicates that the mechanical characteristics
176 of inner steel CHS in this range is analogous to that of a steel CHS under axial compression, as
177 typically shown in Fig. 5(a). This can be explained that, the CFDST with large void ratio has a small
178 amount of the concrete infill, which leads to a limited volume increase of the concrete after crushing,
179 and the outwardly buckled outer tube further reduces the effect of bulky expansion of the crushed
180 concrete on the loading of the inner steel CHS. As can be observed in Fig. 5(b), the concrete infill
181 generally is crushed at the buckling position of both steel CHSs **due to the loss of passive confinement**,
182 and no obvious damage happens to the rest area. These phenomena have also been observed in
183 previous experiments on axially compressed circular-in-circular CFDST stub column specimens with
184 large void ratio [22, 23, 25].

185 The recorded relationship between axial displacement (Δ) and axial load (N) of the specimens are
186 illustrated in Fig. 6 by the solid lines. It can be observed that, all $N - \Delta$ curves develop similarly
187 and generally possess three consecutive phases, namely: approximative elastic, elastoplastic and
188 nonlinear descent after the peak load reached. When other parameters keep constant, the growing in

189 R_v and D_o/t_o produces a reduced initial slope in the approximative elastic phase of the $N - \Delta$
190 curves due to the overall reduction in material area and confinement of outer tube to the concrete
191 infill, and a longer elastoplastic phase of the $N - \Delta$ curves owing to the decrease in ability to resist
192 local instability of inner and outer steel CHS. However, f_{cu} has a moderate influence on the
193 evolvement process of the first two phases of the $N - \Delta$ curves. Moreover, with the variation of
194 three parameters considered in the tests, there is no consistent changing rule in the nonlinear descent
195 phase after the peak load is achieved, which is mainly due to the difference in the final failure patterns
196 and positions of the components in the specimens (see Figs. 4 and 5). In the present research, the
197 ultimate capacity (N_{ue}) is considered as the peak load recorded by the $N - \Delta$ curves, and N_{ue} of
198 all specimens are given in Table 1.

199 Fig. 7 shows axial load (N) versus strain (ε_a and ε_h) relationship of the specimens at two selected
200 cross-sections (S1 and S2), where ε_a and ε_h represent the mean values of axial and hoop strains
201 respectively, and ε_{y_o} is the calculated yield strain of the outer tube and equals to $1.2 f_{y_o}/E_{s_o}$
202 according to the latter material constitutive relationship in the FE model, which is simplified from the
203 measured nominal stress-strain curve, in which E_{s_o} is the modulus of elasticity of the outer tube. It
204 is shown that, the development process of the $N - \varepsilon_a(\varepsilon_h)$ curve is similar to that of the $N - \Delta$
205 curve regardless of the location of SGs, and under the same load level ε_a is larger than ε_h due to
206 the Poisson's effect. For the selected two cross-sections of the same specimen, the $N - \varepsilon_a(\varepsilon_h)$
207 curves possess similar development tendency before reaching N_{ue} ; however, the post-peak stage of
208 the $N - \varepsilon_a(\varepsilon_h)$ curves shows a certain difference due to the discrepancy in the buckling positions of
209 the outer tube. For the same cross-section, the specimen with a larger R_v has a quicker strain
210 development under the same load owing to its lower ultimate capacity, and other two parameters have
211 a moderate impact on the evolvement of the $N - \varepsilon_a(\varepsilon_h)$ curves. The axial strain corresponding to
212 N_{ue} is generally larger than ε_{y_o} , which means that the outer tube shows strength failure rather than
213 local instability failure, although D_o/t_o of groups Aa and Ab is slightly larger than the limit
214 specified in the design guideline [4]. It can also be found that, with the change of experimental

215 parameters, the post-peak phase of the $N - \varepsilon_a(\varepsilon_h)$ curves has no consistent variation rule, as the
216 outer tube buckling positions are not completely located at the sites with strain gauges in the case of
217 the random distribution of material defects.

218 Fig. 8 indicates the variation of ultimate capacity (N_{ue}) of the specimens. It is shown that, with
219 other conditions being the same, R_v , D_o/t_o and f_{cu} all have an impact on N_{ue} of the specimens.
220 N_{ue} of the specimens decrease with the increase of R_v owing to the decrease in area of the concrete
221 infill, and simultaneously the increase of inner tube area is limited. Overall, N_{ue} of the specimens
222 with R_v of 0.85 and 0.9 are 9.9~14.7% and 24.3~33.3% lower than those of the specimens with
223 $R_v=0.8$, respectively. Under the same D_o , the larger the D_o/t_o of the specimens, the lower the
224 ultimate capacity (N_{ue}), because of the relatively bigger area decrease of outer steel CHS, the worse
225 confinement of outer steel CHS to the concrete infill and premature buckling of outer steel CHS with
226 a larger D_o/t_o [23]. Under the same compressive strength of concrete (i.e. $f_{cu} = 49.4$ MPa), the
227 specimens with $D_o/t_o = 143.1$ possess 10.9~16.5% lower N_{ue} values than those with $D_o/t_o =$
228 95.6. Moreover, while D_o/t_o and R_v kept constant, the specimens with a lower f_{cu} show a lower
229 ultimate capacity, and N_{ue} of the specimens with $f_{cu} = 49.4$ MPa are 1.2~15.0% lower than those
230 of the specimens with $f_{cu} = 66.6$ MPa. The above results also indicate that, compared with D_o/t_o
231 and f_{cu} , R_v is the factor that has a greater impact on the ultimate capacity of CFDST stub columns
232 with large void ratio.

233 Similar to the relevant approach in previous studies [25], the elastic compressive stiffness (K_e) of
234 the specimens can be obtained by the following equation:

$$235 \quad K_e = \frac{0.4N_{ue}}{|\varepsilon_{a,0.4}|} \quad (1)$$

236 in which, $\varepsilon_{a,0.4}$ is the measured axial strain corresponding to $0.4N_{ue}$ in the ascending phase of the
237 $N - \varepsilon_a$ curve. The value of K_e is given in Table 1.

238 At the same time, the elastic compressive stiffness of circular-in-circular CFDST cross-section (K_0)
239 used in the design [4, 21] equals to the sum of the elastic compressive stiffness of its three components,
240 and the formula is as follows:

$$K_0 = E_{so} \cdot A_{so} + E_c \cdot A_c + E_{si} \cdot A_{si} \quad (2)$$

where, E_c and E_{si} are the modulus of elasticity of the concrete infill and inner tube, respectively.

The influence of parameters on K_e and D_K is demonstrated in Fig. 9, where D_K is the ratio of K_e to K_0 . The results in Fig. 9 and Table 1 show that, K_e of the specimens generally decrease with the increase of R_v and D_o/t_o and increase with the increase of f_{cu} as the area of both tubes and concrete infill varied with the variation of three parameters. K_e of the specimens with R_v of 0.85 and 0.9 are 7.6~21.2% and 27.4~35.2% lower than those of the specimens with $R_v=0.8$, respectively. In general, the specimens with f_{cu} of 49.4 MPa have 3.5~17.7% lower K_e values than the relevant specimens with $f_{cu} = 66.6$ MPa, and K_e of the specimens with $D_o/t_o = 143.1$ are 12.7~20.8% lower than those of the specimens with $D_o/t_o = 95.6$. Moreover, the effect of three parameters on D_K is similar to their influence on K_e and in general D_K is slightly smaller than unity due to the existence of material defects and variability in material properties. The calculation results show that, D_K varies between 0.807 and 1.025, and the mean and standard deviation of D_K are equal to 0.936 and 0.079, respectively.

3. Finite element (FE) modelling

3.1. Description of the FE model

To study the axial compressive behaviour of circular-in-circular CFDST stub columns with large void ratio numerically, a nonlinear finite element (FE) model was built using ABAQUS [26].

The modulus of elasticity and Poisson's ratio of steel CHSs replicated those acquired from material characteristic tests. The inelastic behaviour of steel CHSs was described by the classical metal plasticity model available in ABAQUS [26]. The relationship between plastic strain and true stress of steel that needs to be imported into the software was obtained based on the nominal one including five phases [7], and the detailed formulae are as follows:

$$\sigma_s = \begin{cases} E_s \cdot \varepsilon_s & (\varepsilon_s \leq \varepsilon_e) \\ -A \cdot \varepsilon_s^2 + B \cdot \varepsilon_s + C & (\varepsilon_e < \varepsilon_s \leq \varepsilon_y) \\ f_y & (\varepsilon_y < \varepsilon_s \leq \varepsilon_q) \\ f_y \cdot \left(1 + 0.6 \frac{\varepsilon_s - \varepsilon_q}{\varepsilon_u - \varepsilon_q}\right) & (\varepsilon_q < \varepsilon_s \leq \varepsilon_u) \\ 1.6f_y & (\varepsilon_s > \varepsilon_u) \end{cases} \quad (3)$$

where, σ_s and ε_s are the nominal stress and strain of steel respectively, $\varepsilon_e=0.8 f_y/E_s$, $\varepsilon_y=1.5\varepsilon_e$, $\varepsilon_q=10\varepsilon_y$, $\varepsilon_u=100\varepsilon_y$, $A = 0.2 f_y/(\varepsilon_y - \varepsilon_e)^2$, $B = 2A \cdot \varepsilon_y$, and $C = 0.8f_y + A \cdot \varepsilon_e^2 - B \cdot \varepsilon_e$.

The size effect of concrete was not considered in the present FE simulation according to the analysis on the experimental data in Fig. 1(b). The modulus of elasticity and Poisson's ratio of the concrete infill were taken as $4730\sqrt{f'_c}$ [27] and 0.2 [28], respectively. The damaged plasticity model in the ABAQUS [26], including isotropic compressive/tensile plasticity as well as isotropic damaged elasticity, was chosen for describing the inelastic property of the concrete infill, in which, the equivalent compressive/tensile plastic strain were adopted to control the yield and failure surface, and the characterization of softening and stiffness deterioration was actualized by the compressive/tensile damage variables, respectively. The tension stiffening of concrete was simulated by the fracture energy cracking criterion [8]. The engineering compressive stress (σ_c) versus strain (ε_c) relationship presented by Wang et al. [7] was adopted to calculate the tabulated data for the compressive stress and the relevant inelastic strain of the concrete infill in a circular-in-circular CFDST, as presented in the following equations:

$$\sigma_c/f'_c = \begin{cases} 2(\varepsilon_c/\varepsilon_{c,p}) - (\varepsilon_c/\varepsilon_{c,p})^2 & (\varepsilon_c/\varepsilon_{c,p} \leq 1) \\ \frac{\varepsilon_c/\varepsilon_{c,p}}{\rho \cdot (\varepsilon_c/\varepsilon_{c,p} - 1)^2 + \varepsilon_c/\varepsilon_{c,p}} & (\varepsilon_c/\varepsilon_{c,p} > 1) \end{cases} \quad (4)$$

where, $\varepsilon_{c,p} = (1300 + 12.5f'_c + 800\xi^{0.2})/1E6$, $\rho = 0.5\sqrt{f'_c} \cdot (2.36E - 5)^{[0.25+(\xi-0.5)^7]} \geq 0.12$, and ξ is the nominal confinement factor [4]. It should be noted that, the adopted engineering compressive $\sigma_c - \varepsilon_c$ relationship took into account the influence of the confinement of outer steel CHS to the concrete infill on the peak strain ($\varepsilon_{c,p}$) and the post-peak stage through variable ξ .

Outer and inner steel CHSs were simulated by S4 elements, which uses the Simpson's rule with 9 integration points in thickness direction, whilst the concrete infill and the steel sleeves (including the

286 stiffeners and endplate on them) were modeled by C3D8R elements. In this study, the structured
287 meshing technology available in ABAQUS was selected while the meshing size about $D_o/12$, and
288 both steel CHSs and the concrete infill had the same mesh nodes, so as to guarantee the deformation
289 coordination of different components of the composite columns and to improve the convergence of
290 the FE modelling. The meshing used in the FE model is shown in Fig. 10.

291 The surface to surface contacts were considered in simulating the interface features between
292 different components of the FE model. To model the interface between steel CHSs and the concrete
293 infill, the 'hard contact' was used in the normal direction, which enables the compressive stress
294 perpendicular to the contact surfaces to be transferred completely between the interfaces, and the
295 contact surfaces between different components can be separated but not penetrated. At the same time,
296 the 'Coulomb friction' model was used in the tangential directions and the friction coefficient was
297 equal to 0.6 according to previous simulations [7, 8], i.e. the interfaces can transfer shear stresses,
298 and relative sliding between the interfaces occurs after the shear stress reaches a critical value [26].
299 For the interface between the endplate on the sleeves and the concrete infill, only the 'hard contact'
300 constraint in the normal direction was defined. Moreover, the 'shell-to-solid coupling' constraint was
301 considered for the interface between the endplate on the sleeves and both steel CHSs, and the interface
302 between the sleeves and outer steel CHS was defined as the 'Tie' constraint.

303 The FE model of the CFDST stub columns together with the sleeves was constructed using the
304 ABAQUS Standard [26], and the boundary conditions are indicated in Fig. 10. Two reference points
305 respectively coupled with the upper and lower endplate on the steel sleeve were set at the center of
306 the endplates to define the boundary conditions. The 'ENCASTRE' was set to the reference point of
307 the lower endplate, i.e. all degrees of freedom were constrained, and the reference point of the upper
308 endplate has no translational displacement in X and Y directions, i.e. $U_x=U_y=0$. During the loading
309 step, axial displacements along Z direction were applied to the reference point of the upper endplate.

310 As is well known, the influence of initial imperfections and residual stresses on the performance
311 of steel tubes is evident. However, the investigations of Wang et al. [7] showed that, compared with

312 the steel CHSs, the effects of initial imperfections and residual stresses on circular CFDST stub
313 columns were reduced by the concrete infill significantly, and thus the initial imperfections and
314 residual stresses were not incorporated into the current FE modelling.

315 **3.2. Verification of the FE model**

316 Typical failure patterns of the specimens predicted by the FE model are demonstrated in Fig. 11. From
317 the comparison between Fig. 11 and Figs. 4 and 5, it can be observed that, the FE results generally
318 reflect the failure characteristics of the whole composite column and its components, i.e. local
319 buckling of outer and inner tube along the circumference appear at 1~2 regions, while the concrete
320 infill deforms obviously at the local buckling regions of both tubes irrespective of R_v value.
321 However, the predicted local buckling and deformed regions are different from the experimental
322 observations, and there is no evident rotation for the top and bottom surface of the components. This
323 is attributed to the randomness of material defect distribution in the specimens and the inclining of
324 top endplate to the severely damaged region of the specimens, and these cannot be considered in the
325 current FE model.

326 The simulated $N - \Delta$ curves are compared with the recorded ones in Fig. 6. It is shown that,
327 analogous to the curve of the specimens, the simulated $N - \Delta$ curve also consists of approximative
328 elastic, elastoplastic and nonlinear post-peak stages. However, the simulated $N - \Delta$ curves possess
329 a higher initial slope, and a slower bearing capacity descending and a more stable residual bearing
330 capacity after achieving the peak load. It should be noted that, there are several facts in the specimens
331 that cannot be reflected in the FE model, mainly including randomly distributed material defects,
332 variation of material properties, clearance between steel CHSs and the concrete infill, possible non-
333 axial compression after the peak load, etc. These cause the abovementioned disparity between two
334 kinds of $N - \Delta$ curves. The predicted $N - \varepsilon_a(\varepsilon_h)$ relationship at cross-section S1 are in
335 comparison with the measured results in Fig. 12, where the letters ‘M’ and ‘P’ in the parentheses
336 respectively denote the measured and predicted results. It is shown that, the predicted elastic stage of
337 $N - \varepsilon_a(\varepsilon_h)$ curves is generally in good agreement with the measured one; however, the elastoplastic

338 and post-peak stage of the predicted curves show a certain difference with the corresponding stages
339 of the measured results, considering that there is discrepancy between the predicted bucking positions
340 of both tubes and the measured results, as shown in Figs. 4, 5 and 11.

341 The deviation between the predicted ultimate capacities by the FE model (N_{ufe}) and the measured
342 results (N_{ue}) in this and the previous experiments is indicated in Fig. 13. In the FE simulation,
343 polymer concrete and grout adopted in the previous tests [22, 24, 25] are temporarily treated as
344 ordinary concrete, considering that there is currently no mature constitutive model for these two kinds
345 of concrete. N_{ufe} of the specimens in this study are given in Table 1. An analysis of all 27 data in
346 Fig. 13 demonstrates that, the mean and standard deviation of N_{ufe}/N_{ue} respectively equal to 0.946
347 and 0.050, and the predicted ultimate capacities are generally limited to 10% of the measured results.

348 The above comparison and analysis indicate that, the FE model developed in this study is generally
349 accurate to investigate the axial compressive behaviour of circular-in-circular CFDST stub columns
350 with large void ratio.

351 **4. Parametric study**

352 The impact of factors on stress state of each component of the CFDST stub columns with large void
353 ratio while reaching the ultimate capacity was further investigated by the validated FE model. The
354 basic conditions of the computing examples included: $D_o = 540$ mm, $H = 1500$ mm, $D_i/t_i = 60$,
355 $f_{yo} = f_{yi} = 355$ MPa, $f'_c = 50$ MPa, $R_v = 0.8\sim 0.95$, and nominal steel ratio $\alpha_n = 0.08$, in which
356 α_n equals to the ratio of A_{so} to the cross-sectional area enclosed by the inner wall of the outer steel
357 CHS (A_{ce}) [4]. In addition, the corresponding circular CFST with the same material and geometric
358 properties of the outer steel CHS as the circular-in-circular CFDST was also taken into account. The
359 Mises stress of steel CHSs and the longitudinal stress (S33) of concrete at the mid-height section were
360 obtained and analyzed.

361 The comparison of the Mises stress of outer steel CHS between CFDST and CFST is indicated in
362 Fig. 14. It is shown that, the stress distribution of outer tube in the CFDST is the same as that of steel
363 tube in the CFST, and the Mises stress in most areas of outer tube in the CFDST and CFST reaches

364 f_{y0} . The distribution of the Mises stress of inner steel CHS in the CFDST is similar to that of outer
365 steel CHS, and f_{yi} can also be reached in most areas of inner tube in the CFDST, as shown in Fig.
366 15. The influence of parameters on $\sigma_{si,u}/f_{yi}$ of inner steel CHS is plotted in Fig. 16, in which $\sigma_{si,u}$
367 is the maximum Mises stress. It can be seen that, all parameters have a moderate effect on $\sigma_{si,u}/f_{yi}$,
368 and $\sigma_{si,u}$ is slightly higher than f_{yi} .

369 The variation of the longitudinal stress (S_{33}) of concrete in the CFDST with different void ratio
370 and the CFST is demonstrated in Fig. 17. It is shown that, the S_{33} of all sections are larger than f'_c ;
371 however, the S_{33} of CFDST columns are smaller than those of the corresponding CFST columns as
372 the confinement effect of outer tube on the concrete is reduced by the inward deformation of the
373 concrete infill and inner tube. Furthermore, the S_{33} of CFDST columns is small on the inside and
374 large on the outside; however, the S_{33} of CFST columns is large on the inside (center) and small on
375 the outside (edge). This is attributed to the fact that, under the same axial deformation, the inner tube
376 of CFDST columns deforms inwards due to the Poisson's effect and the void characteristics, which
377 results in a weaker supporting action of inner steel CHS on the transverse deformation of concrete
378 compared with the confinement effect of outer tube, and thus, the transverse confinement of the
379 concrete infill is gradually reduced from the outside to the inside. However, the closer the core
380 concrete of CFST columns is to the center, the stronger the transverse deformation is constrained by
381 its peripheral concrete and the outer tube.

382 Fig. 18 indicates the effect of parameters on S_{33}/f'_c of the concrete infill in the CFDST, where
383 D_c is the thickness of the concrete infill. It can be discovered that, in generally, R_v , α_n , f_{y0} and f'_c
384 have significant effect on S_{33}/f'_c , and D_i/t_i has a moderate influence on S_{33}/f'_c . A bigger S_{33}/f'_c
385 is caused with the increase of α_n and f_{y0} and decrease of f'_c . In addition, due to the difference
386 between the supporting action of inner tube and the confinement effect of outer tube to the concrete
387 infill, S_{33}/f'_c on the side near the inner edge increases with the increase of R_v , whilst S_{33}/f'_c on
388 the side near the outer edge decreases with the increase of R_v .

389 **5. Ultimate capacity prediction**

390 Based on the results in this study (see Figs. 15 and 16) and the suggestions in the literature [4], it
 391 is assumed that, the axial stress of inner steel CHS equals to its yield strength (f_{yi}) when reaching the
 392 ultimate capacity of axially compressed circular-in-circular CFDST stub columns with large void
 393 ratio. As a result, the composite strength index ($f_{scy,v}$) can be defined as follows:

$$394 \quad f_{scy,v} = \frac{N_u - f_{yi} \cdot A_{si}}{A_{so} + A_c} \quad (5)$$

395 where, N_u is the ultimate capacity obtained by the FE simulation.

396 The FE modelling results show that, the parameters that have an important effect on $f_{scy,v}$ include
 397 R_v , α_n , f_{yo} and f'_c , as indicated by the solid lines in Fig. 19, and $f_{scy,v}$ augments with the increase
 398 of R_v , α_n and f_{yo} and decrease of f'_c . By regressing the data in Fig. 19 and referring to the
 399 formulae of such composite columns with small void ratio [4], it is found that, in general, the formula
 400 that applies to circular-in-circular CFDST stub columns with $R_v \leq 0.75$ can also be applied to those
 401 with large void ratio, and the equation is:

$$402 \quad f_{scy,v} = \frac{\alpha}{1+\alpha} \cdot R_v^2 \cdot f_{yo} + \frac{1+\alpha_n}{1+\alpha} \cdot (1.14 + 1.02\xi) \cdot f_{ck} \quad (6)$$

403 in which, α ($=A_{so}/A_c$) is the steel ratio, and f_{ck} is the characteristic compressive strength of
 404 concrete [4].

405 Fig. 19 indicates the comparison between the simplified and numerical $f_{scy,v}$. It is shown that, the
 406 simplified results accord well with the numerical ones, which indicates that Eq. (6) can predict the
 407 composite strength index of circular-in-circular CFDST stub columns with large void ratio well.
 408 Therefore, by substituting Eq. (6) into Eq. (5), the model for the ultimate capacity prediction of axially
 409 compressed circular-in-circular CFDST stub columns with large void ratio can be obtained:

$$410 \quad N_u = \left[\frac{\alpha}{1+\alpha} \cdot R_v^2 \cdot f_{yo} + \frac{1+\alpha_n}{1+\alpha} \cdot (1.14 + 1.02\xi) \cdot f_{ck} \right] \cdot (A_{so} + A_c) + f_{yi} \cdot A_{si} \quad (7)$$

411 The influence of R_v on N_{us}/N_{ue} of circular-in-circular CFDST stub columns with large void
 412 ratio is plotted in Fig. 20, where N_{us} is the simplified ultimate capacity based on Eq. (7), and a total
 413 of 27 data from the literature and this study are covered. The results indicate that, the minimum and
 414 maximum values of N_{us}/N_{ue} are 0.851 and 1.044, respectively, while the mean and standard

415 deviation are 0.939 and 0.051, respectively. As a result, the simplified model is suitable for the
416 ultimate capacity prediction of axially compressed circular-in-circular CFDST stub columns with
417 void ratio extended to 0.95 and generally tends to be safe. The range of valid parameters applicable
418 to Eq. (7) is: $R_v = 0\sim 0.95$, $\alpha_n = 0.04\sim 0.12$, $f_{y0}(f_{yi}) = 235\sim 460$ MPa, $f'_c = 25\sim 75$ MPa and
419 $D_i/t_i = 30\sim 90$.

420 **6. Conclusions**

421 According to the experimental study and finite element (FE) simulation on axial compressive
422 behaviour of circular-in-circular CFDST stub columns with large void ratio presented in this study,
423 the following conclusions can be achieved:

424 (1) After the tests completed, the outer steel CHS mainly buckles outward at the region near the
425 sleeve, while the outer tube of the specimens with a smaller D_o/t_o and R_v of 0.8 and 0.9 also
426 buckles outward at the mid-height region, and a smaller D_o/t_o leads to a smaller outward buckling
427 deformation. Simultaneously, the inner steel CHS buckles inward and/or outward along the height
428 direction within the buckling range of the outer tube. Moreover, at the primary buckling area of both
429 steel CHSs, crushing of the concrete infill appears.

430 (2) Generally, there are three key stages in the $N - \Delta(\varepsilon)$ curve of the specimens, namely:
431 approximative elastic, elastoplastic and nonlinear post-peak. R_v and D_o/t_o have obvious effect on
432 the ascending stage of the curve, and all three parameters have no consistent effect on the nonlinear
433 post-peak stage of the curve due to the difference in the final failure positions and patterns of the
434 components. Moreover, a higher R_v results in a quicker strain development, and other two
435 parameters have a moderate impact on the evolvement of the $N - \varepsilon$ curves.

436 (3) While other parameters kept constant, N_{ue} and K_e of circular-in-circular CFDST stub
437 column specimens with large void ratio decrease with the augment of R_v and D_o/t_o and the reduce
438 of f_{cu} . $N_{ue}(K_e)$ of the specimens with R_v of 0.85 and 0.9 are 9.9~14.7% (7.6~21.2%) and
439 24.3~33.3% (27.4~35.2%) lower than those of the specimens with $R_v=0.8$ respectively, and the
440 specimens with a larger D_o/t_o and a lower f_{cu} possess 10.9~16.5% (12.7~20.8%) and 1.2~15.0%

441 (3.5~17.7%) lower $N_{ue}(K_e)$, respectively. Furthermore, the measured elastic compressive stiffness
442 of the specimens are generally close to the calculated values according to the design method.

443 (4) The FE model built using the ABAQUS is generally accurate to predict the failure patterns, the
444 load versus displacement (strain) curves and the ultimate capacity of axially compressed circular-in-
445 circular CFDST stub column specimens with large void ratio.

446 (5) Based on the stress distribution characteristics of both steel CHSs and the concrete infill
447 simulated by the FE model, the calculation model for the ultimate capacity of axially compressed
448 circular-in-circular CFDST stub column specimens with large void ratio is developed, from which
449 the calculated ultimate capacities accord well with the measured results.

450 It is apparent that the CFDST members may be subjected to unbalanced bending moments in
451 addition to axial forces. In the future, further investigations into the performance of CFDST beams
452 and beam-columns with large void ratio are needed to guide the design and application of such new
453 composite sections.

454 **Declaration of Competing Interest**

455 The authors declare that they have no known competing financial interests or personal relationships
456 that could have appeared to influence the work reported in this paper.

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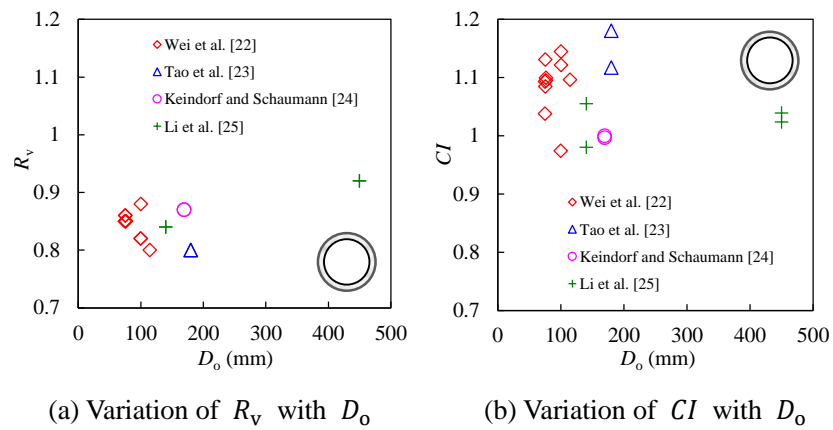
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Figures:



(a) Variation of R_v with D_o

(b) Variation of CI with D_o

Fig. 1. Test data in the literature.

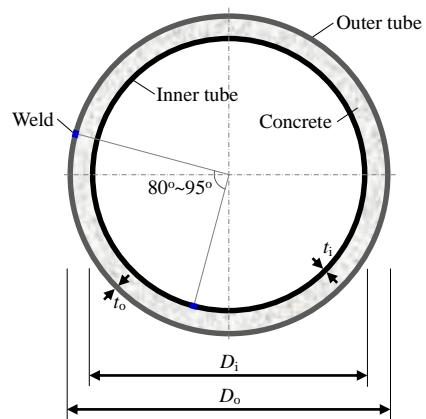


Fig. 2. Cross-section of the specimens.

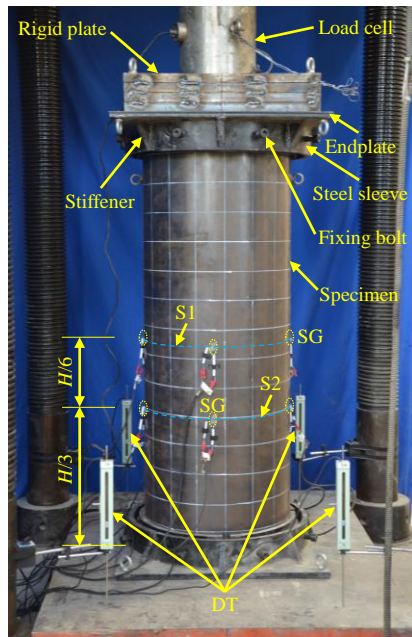
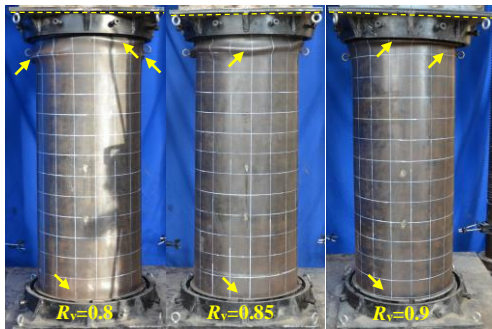
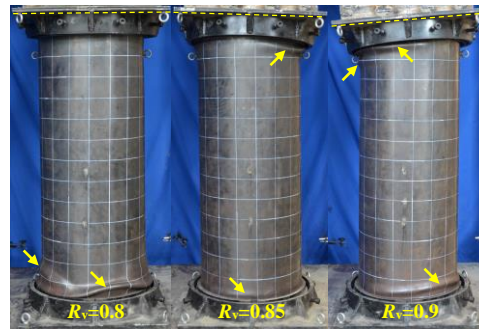


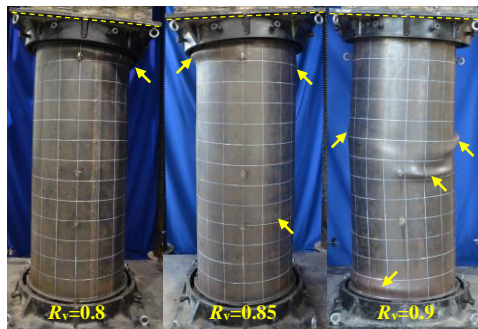
Fig. 3. Picture of test set-up and instrumentations.



(a) Group Aa

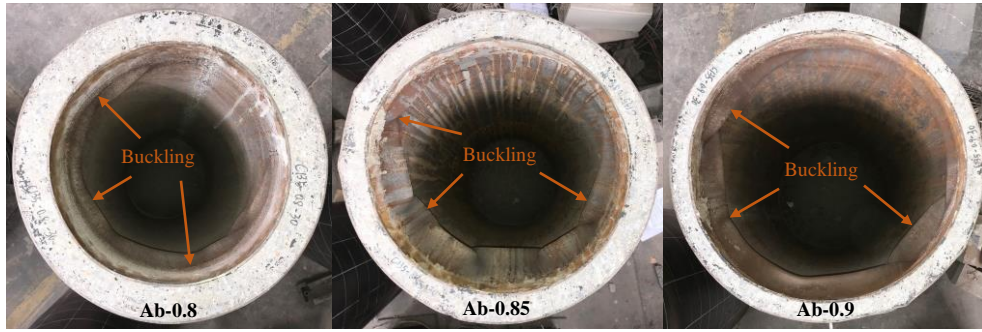


(b) Group Ab

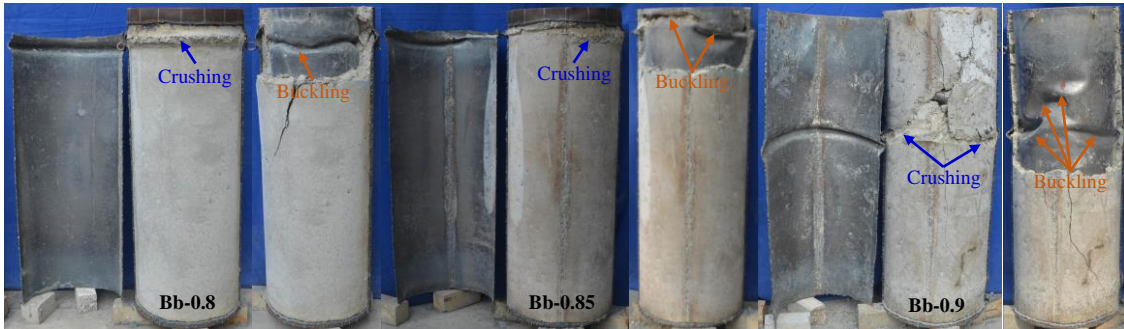


(c) Group Bb

Fig. 4. Failure pattern of the specimens.

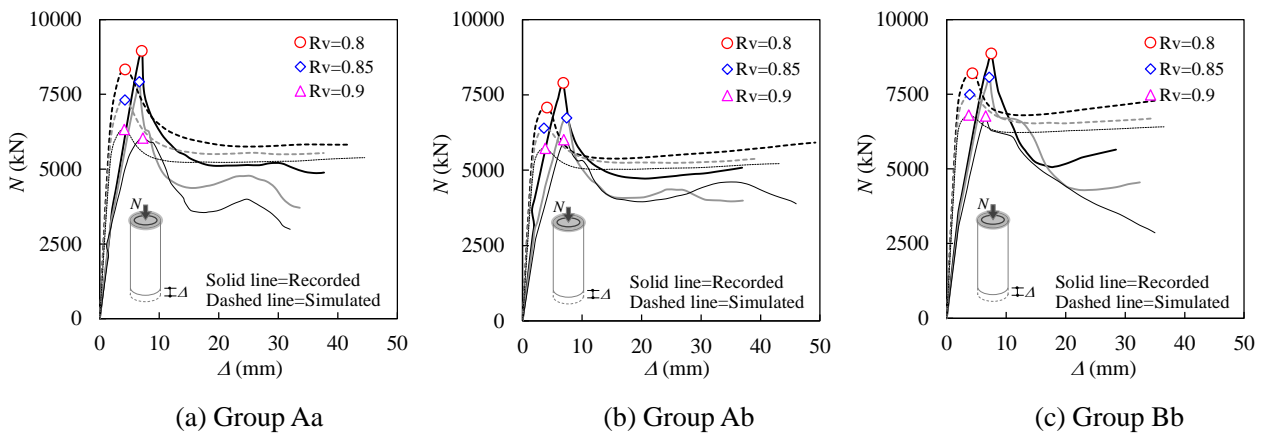


(a) Inner steel CHS



(b) The concrete infill

Fig. 5. Typical failure pattern of two internal components.



(a) Group Aa

(b) Group Ab

(c) Group Bb

Fig. 6. $N - \Delta$ relationship of the specimens.

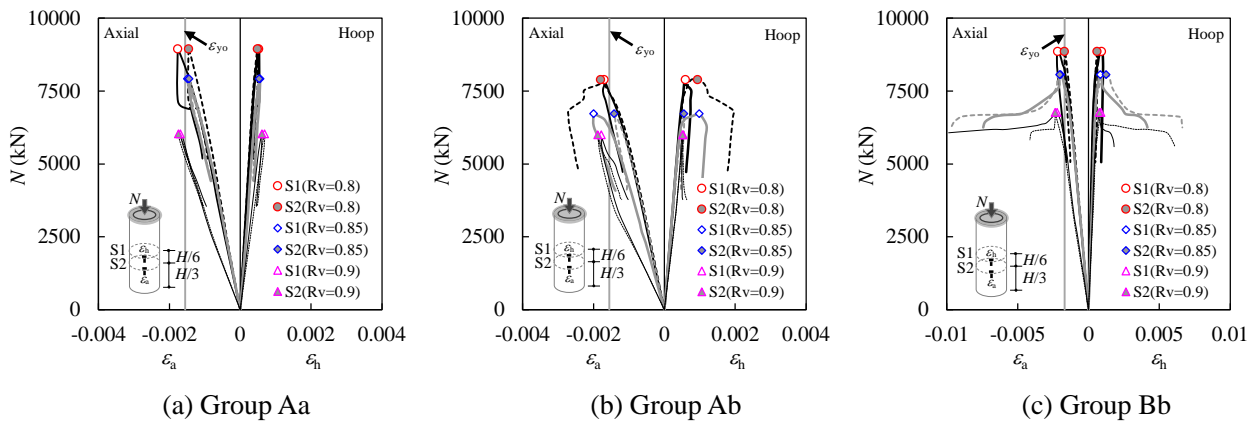


Fig. 7. Axial load (N) versus strain (ε_a and ε_h) relationship of the specimens.

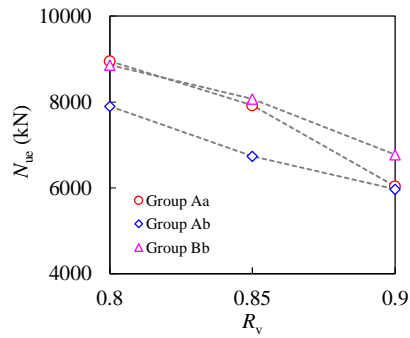


Fig. 8. Variation of N_{ue} .

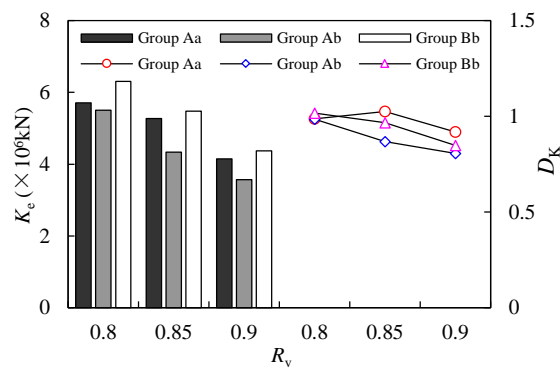


Fig. 9. Influence of parameters on K_e and D_k .

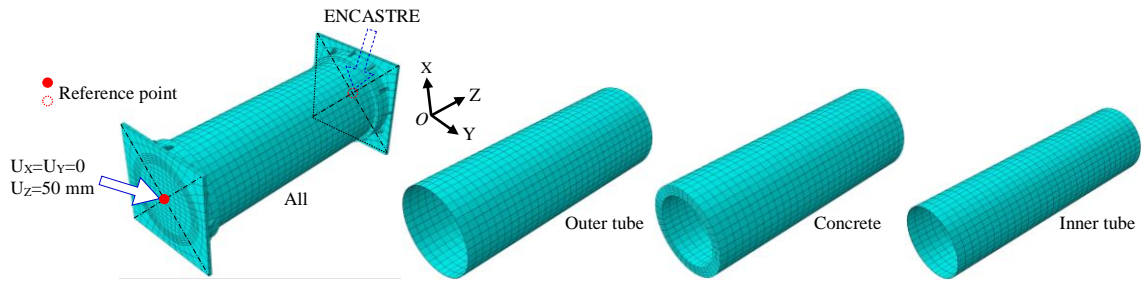


Fig. 10. Meshing and boundary conditions of the FE model.

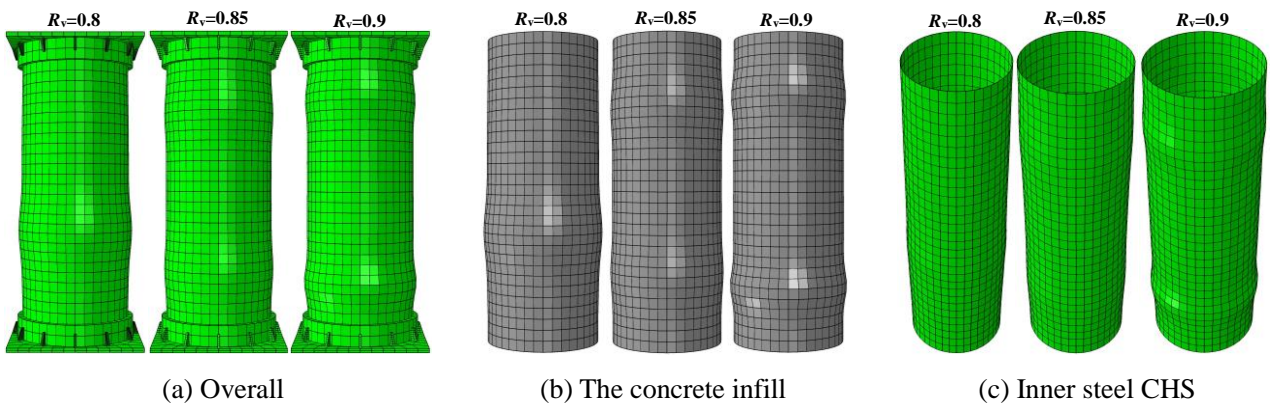


Fig. 11. Typical failure patterns of the specimens predicted by the FE model.

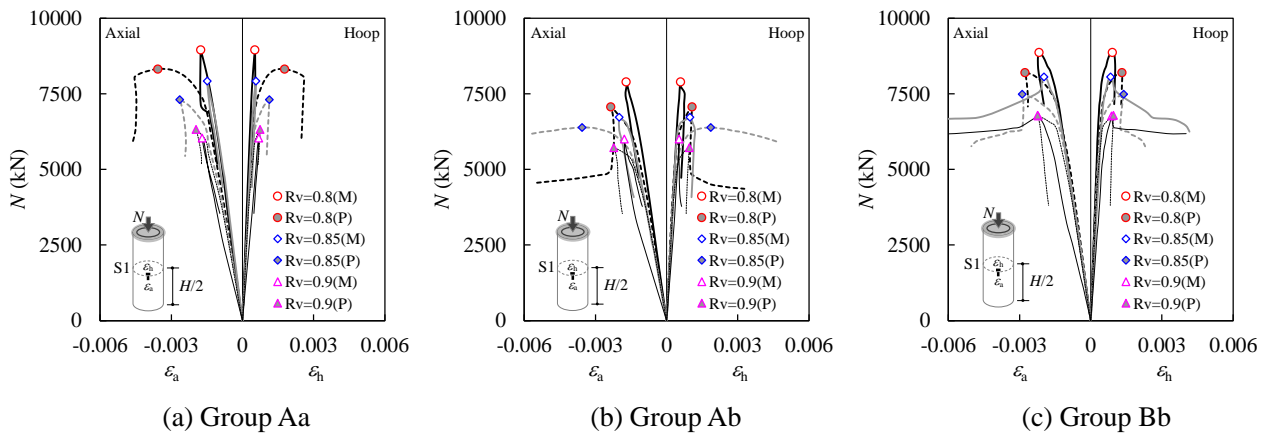


Fig. 12. Comparison between the predicted and measured $N - \varepsilon_a(\varepsilon_h)$ relationship at section S1.

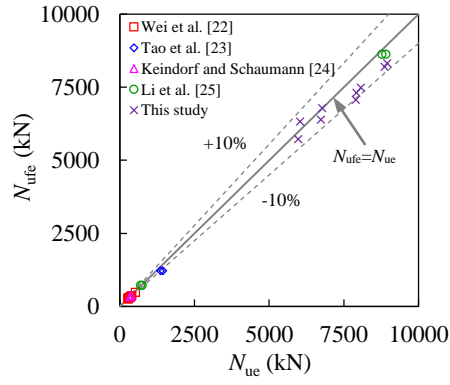


Fig. 13. Deviation between the predicted and measured ultimate capacities.

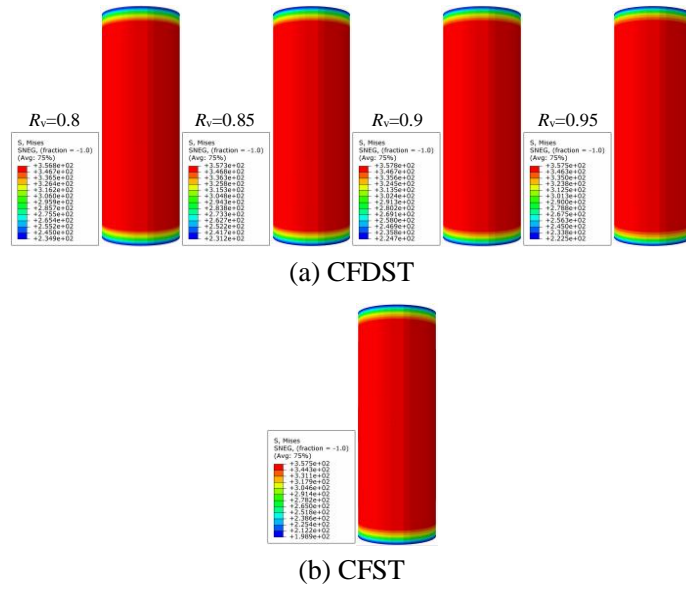


Fig. 14. Comparison of the Mises stress of outer tube between CFDST and CFST.

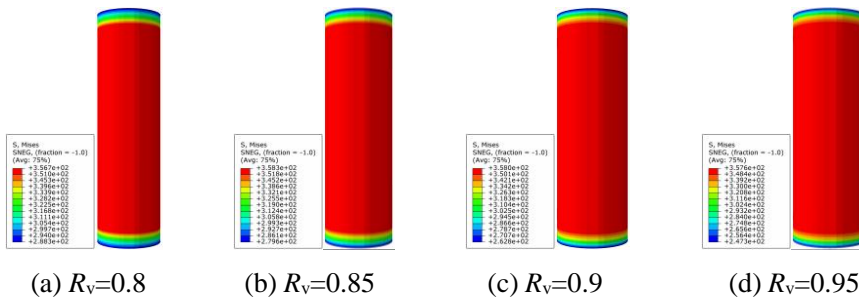


Fig. 15. The Mises stress of inner tube of CFDST.

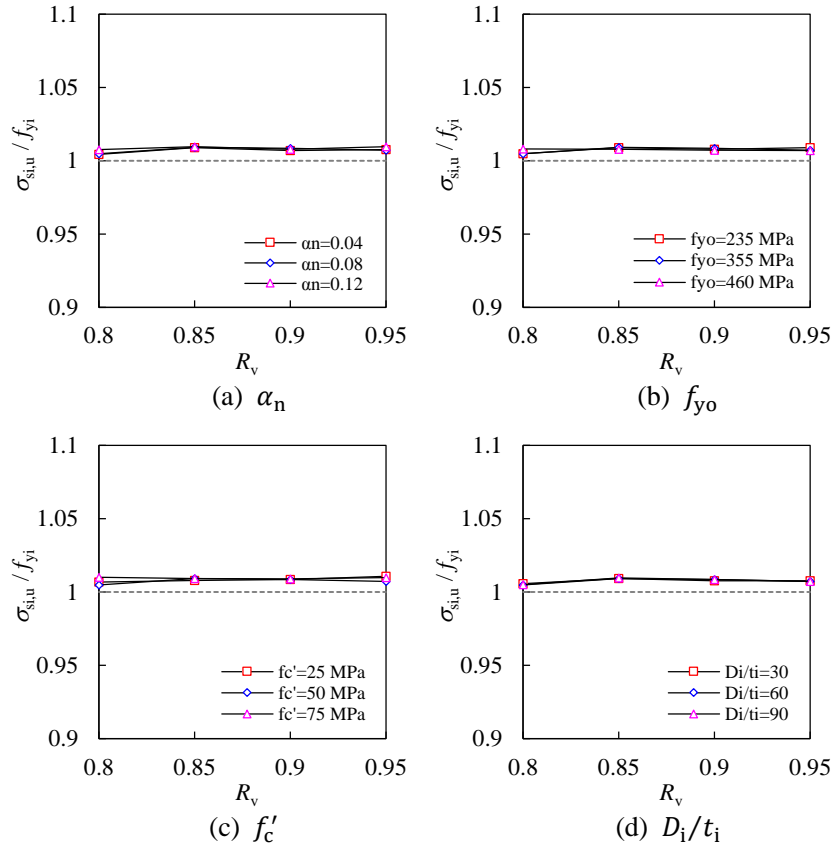


Fig. 16. Influence of parameters on $\sigma_{si,u}/f_{yi}$ of inner steel CHS.

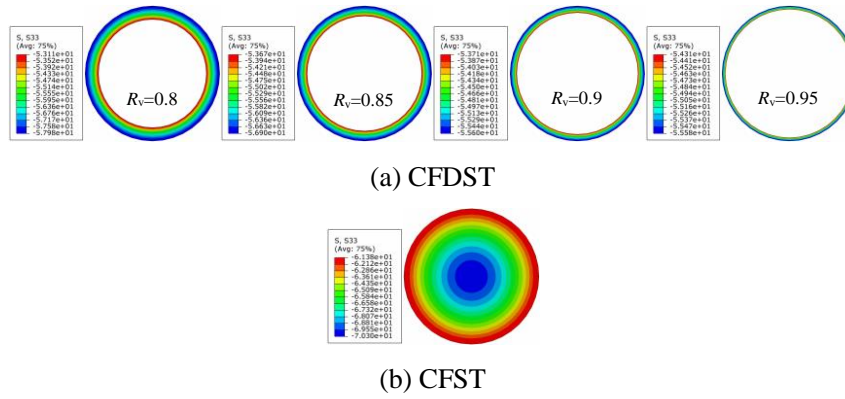


Fig. 17. Variation of the longitudinal stress (S33) of concrete.

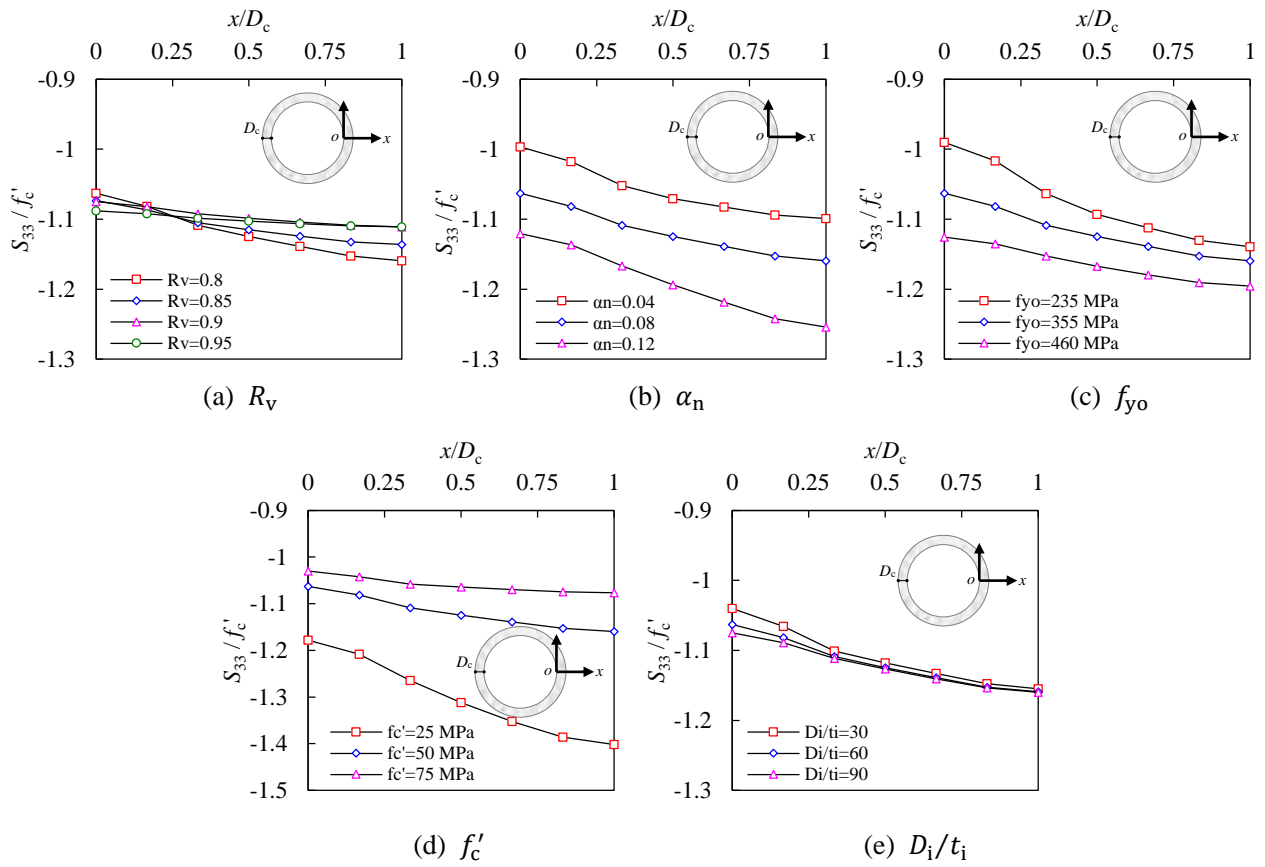


Fig. 18. Effect of parameters on S_{33}/f'_c of the concrete infill in the CFDST.

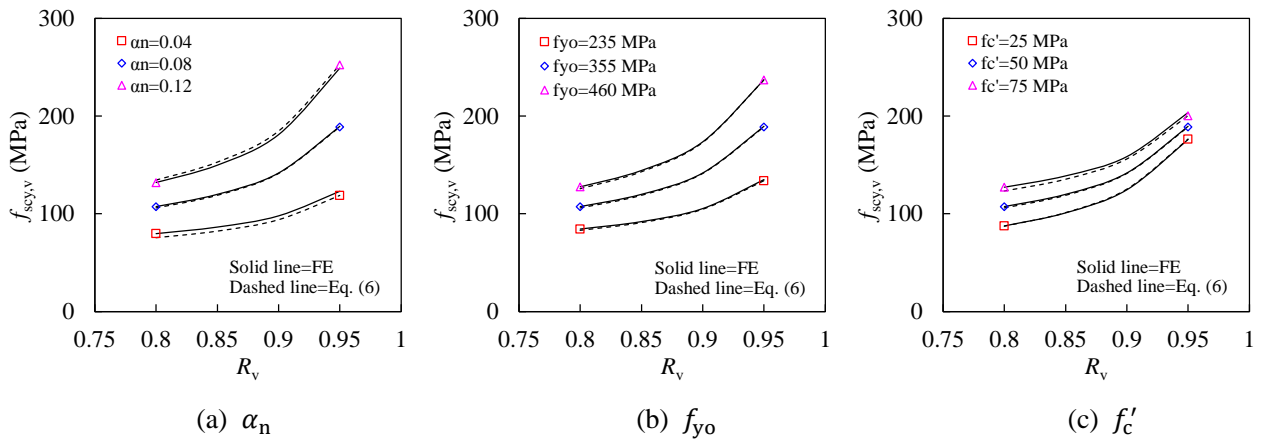


Fig. 19. Effect of parameters on composite strength index ($f_{scy,v}$).

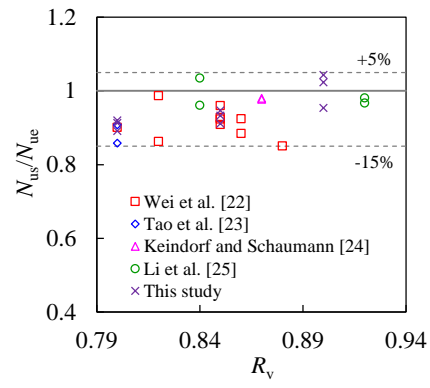


Fig. 20. Influence of R_v on N_{us}/N_{ue} .

Tables:**Table 1.** Information of the specimens

No.	Label	$D_o \times t_o$ mm×mm	$D_i \times t_i$ mm×mm	D_o/t_o	D_i/t_i	R_v	f_{y0} (MPa)	f_{yi} (MPa)	f_{cu} (MPa)	K_c ($\times 10^6$ kN)	N_{ue} (kN)	N_{ufe} (kN)	$\frac{N_{ufe}}{N_{ue}}$
1	Aa-0.8	538×3.76	418×5.63	143.1	74.2	0.8	253.8	296.3	66.6	5.707	8949.7	8321.3	0.930
2	Aa-0.85	538×3.76	449×5.63	143.1	79.8	0.85	253.8	296.3	66.6	5.272	7924.7	7310.8	0.923
3	Aa-0.9	538×3.76	477×5.63	143.1	84.7	0.9	253.8	296.3	66.6	4.145	6036.7	6329.8	1.049
4	Ab-0.8	538×3.76	418×5.63	143.1	74.2	0.8	253.8	296.3	49.4	5.505	7896.6	7070.8	0.895
5	Ab-0.85	538×3.76	449×5.63	143.1	79.8	0.85	253.8	296.3	49.4	4.337	6735.2	6391.6	0.949
6	Ab-0.9	538×3.76	477×5.63	143.1	84.7	0.9	253.8	296.3	49.4	3.565	5966.8	5725.8	0.960
7	Bb-0.8	538×5.63	420×5.63	95.6	74.2	0.8	296.3	296.3	49.4	6.306	8864.0	8199.6	0.925
8	Bb-0.85	538×5.63	448×5.63	95.6	79.8	0.85	296.3	296.3	49.4	5.478	8068.0	7487.0	0.928
9	Bb-0.9	538×5.63	473×5.63	95.6	84.7	0.9	296.3	296.3	49.4	4.367	6774.0	6795.3	1.003

Table 2. Properties of steel

Type	Thickness (mm)	f_y (MPa)	f_u (MPa)	E_s ($\times 10^5$ N/mm ²)	μ_s	δ_{ef} (%)
A	3.76	253.8	395.7	1.96	0.289	18.0
B	5.63	296.3	420.2	2.11	0.279	16.3

Table 3. Mix proportions and properties of the concrete

Type	Mix proportions (kg/m ³)						Properties			
	Cement	Fly ash	Coarse aggregate	Sand	Water	WRA*	$f_{cu,28}$ (MPa)	f_{cu} (MPa)	E_c (GPa)	Slump (mm)
a	420	130	832	800	189.5	11.62	54.2	66.6	35.9	270
b	325	208	911	790	103	7.20	30.6	49.4	33.6	245

*WRA=water-reducing admixture.