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# Static performance of square CFDST chord to steel SHS brace T-joints

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**Abstract:** In this paper, static performance of square concrete-filled double-skin steel tube (CFDST) chord to steel square hollow section (SHS) brace T-joints are investigated through experimental and numerical studies. Twelve specimens, including 8 T-joints with square CFDST chord, 2 T-joints with square concrete-filled steel tube (CFST) chord and 2 T-joints with steel SHS chord as counterparts, were tested under continuously increasing compressive force on the brace with concentric compression load applied simultaneously to the chord. The influence of chord type, brace-to-chord width ratio ( $\beta$ ) and concentric compression level of the chord ( $n$ ) on the static performance of the T-joints was examined. It is found that the composite T-joints have enhanced static performance than their steel counterparts. For the composite T-joints, the failure pattern varies from compression-flexure-shear failure of composite chord to local buckling of steel SHS brace when  $\beta$  reduces. Moreover, while only the composite chord failure occurs, the bearing capacity of the specimens augments with growing of hollow ratio of CFDST chord ( $\chi$ ),  $\beta$  and  $n$ ; however, when only steel SHS brace of the composite joints fails, the chord type has a moderate influence on the bearing capacity of the specimens. The static performance of the T-joints was simulated using a finite element (FE) model, which is validated against the observations in the experiment. On the basis of the experimental and numerical research, the design formulae for bearing capacity of the composite T-joints were developed, and a good accuracy of the computations was achieved.

**Key words:** T-joint; square hollow section (SHS); concrete-filled double-skin steel tube (CFDST); static tests; finite element simulation; bearing capacity computation

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## 32 **1. Introduction**

33 Due to the excellent structural performance and attractive aesthetic effect, steel hollow section trussed  
34 structures are adopted extensively in buildings, bridges, offshore structures, towers and masts, and so  
35 on [1], in which the joints are subjected to the most complicated loadings, causing complicated  
36 interaction between chords and braces. Generally, the chords and braces in a steel hollow section truss  
37 are connected through welding; however, the welded joints may prone to fail due to their low strength  
38 and poor fatigue resistance caused by the severe concentration of stress in welding zone as well as  
39 the existence of the initial defects of welding. To improve the structural performance of the welded  
40 joints between steel hollow section chords and braces, Packer [2] first proposed a new kind of  
41 composite tubular joints with the chord fully or partially filled with concrete, i.e. steel hollow section  
42 brace(s) to concrete-filled steel tube (CFST) chord joints. However, when the outer size of the chord  
43 is increased, the filling concrete will restrict the practical application of the composite tubular joints  
44 owing to the increased weight of the structure and the labor cost. In such a case, replacing CFST with  
45 the novel concrete-filled double-skin steel tube (CFDST) becomes a good choice. Generally, CFDST,  
46 which is composed of two concentric steel tubes and concrete between them, is a sort of new  
47 composite member originated from the CFST, and through reasonable design the composite tubular  
48 joints with CFDST chords can be lighter than those with CFST chords while maintaining the similar  
49 performance [3-5]. [Typical on-sight photo and schematic view of composite tubular joints are](#)  
50 [demonstrated in Fig. 1.](#)

51 The earliest research towards the static property of the composite T- and K-joints using rectangular  
52 CFST chord and steel hollow section brace was reported by Packer [2], where three tension-loaded  
53 T-joints and six gapped K-joints were tested and the results indicated that the composite tubular joints  
54 had a significant different failure pattern and a superior strength relative to their counterparts with  
55 steel hollow section chord and brace(s). Since then, some researchers started to pay attention to the  
56 structural or fatigue behaviour of the composite tubular joints, including experimental investigation  
57 into circular CFST chord to steel circular hollow section (CHS) brace T-joints subjected to uniplanar

58 monotonic and cyclic bending at the brace end with axial compression applied to the chord [6,7] and  
59 cyclic axial loading upon the brace [8], CFST T- and X-joints manufactured from stainless steel  
60 square hollow section (SHS) and rectangular hollow section (RHS) subjected to concentric  
61 compression at one brace end [9, 10], square CFDST chord to steel SHS brace X-joints under axial  
62 compression upon braces [11], circular CFDST chord to steel CHS brace(s) T- and K-joints under  
63 static loading [12, 13], full-scale K-joints adopting steel CHS brace and circular CFST chord under  
64 static loading [14], square CFST chord to steel SHS brace T-joints subjected to uniplanar fatigue  
65 loading upon the brace [15], circular CFST chord to steel CHS brace X-joints with axial loading  
66 applied to the chord [16], unreinforced and reinforced circular CFST chord to steel CHS brace K-  
67 joints with axial tensile load upon the chord [17-19], and T-, Y-, K-, and KT-joints using steel CHS  
68 brace and circular CFST chord with axial tensile load upon one brace [20, 21], and numerical  
69 simulation of T- and K-joints using steel CHS brace and circular CFDST chord under static loading  
70 [12, 22], steel CHS braces to circular CFST chord N-joints with axial loading applied to the brace  
71 end [23], circular CFST chord to steel CHS brace T-joints with axial loading acting on the brace end  
72 [24, 25], unreinforced and reinforced circular CFST chord to CHS steel brace K-joints with axial  
73 tensile load upon the chord [18, 19], and T-, Y-, K-, and KT-joints using steel CHS brace and circular  
74 CFST chord with axial tensile load upon one brace [26]. Furthermore, the simplified formulae for the  
75 static strength of K-joints using steel CHS brace and circular CFDST chord [22], the stress  
76 concentration factors (SCFs) of T- and N-joints using circular CFST chord and steel CHS brace with  
77 axial loading applied to the end of the braces [23, 25], and the ultimate strength of circular CFST  
78 chord to steel CHS brace T- and Y-joints with chord broken by punching and shearing [26], have also  
79 been recommended. Previous studies have revealed that: 1) compared to the corresponding steel  
80 tubular joints the composite tubular joints possess different failure patterns and higher strength and  
81 stiffness, 2) filling concrete into the chord greatly enhances the seismic performance of the steel  
82 hollow section joints, 3) the welded composite tubular joints generally have lower peak SCFs and  
83 better fatigue strength than the steel counterparts, and 4) the existing design method for the steel

84 tubular joints cannot be directly applied to the design of composite joints.

85 It is also noticeable that, only few researchers [2, 9-11, 15] carried out the study on the static or  
86 fatigue performance of the composite T-, K- and X-joints using square/rectangular CFST/CFDST  
87 chord, and developed the static strength calculation formulae or fatigue design method of such kind  
88 of joints. These studies provide the basis for the research of the composite T-joints having square  
89 CFDST chord with pined-pined ends. Therefore, this paper tries to experimentally and numerically  
90 investigate the behaviour of square CFDST chord to steel SHS brace T-joints under static loading.  
91 The design formulae for the bearing capacity of such composite joints were eventually put forward  
92 according to the results of systematic finite element simulation.

## 93 **2. Experimental investigation**

### 94 **2.1. Material properties**

95 Standard tensile coupon tests were performed to measure the properties of steel, and the results are  
96 given in Table 1. According to EC3 [27] and ANSI/AISC-360 [28], the SHS with width and thickness  
97 of 200 mm and 4.00 mm belongs respectively to the Class 4 and non-compact section, i.e. local  
98 buckling will occur before achieving its yield strength in one or more parts of the cross-section, whilst  
99 other steel SHSs belong to the compact section. Furthermore, according to [5], the SHS with width  
100 and thickness of 200 mm and 4.00 mm will not buckle in advance due to the supporting effect of the  
101 sandwiched concrete when it is used as the outer tube of CFDST.

102 Concrete mix with design strength class C45 was poured into the sandwich between two tubes or  
103 the outer tube of the composite chords. The mix-proportion of the concrete was as follows: P.O42.5  
104 cement, 420 kg/m<sup>3</sup>; the first grade fly ash, 130 kg/m<sup>3</sup>; limestone rubble with particle size of 5-10 mm,  
105 832 kg/m<sup>3</sup>; river sand, 800 kg/m<sup>3</sup>; tap water, 189.5 kg/m<sup>3</sup>; and Polycarboxylate water reducing agent,  
106 6.88 kg/m<sup>3</sup>. The fresh concrete had a slump of 270 mm and a spreading of 661 mm. To measure the  
107 compressive strength and modulus of elasticity of the concrete, a number of cubes with 150 mm side  
108 length and prisms with dimensions of 150 mm×150 mm×300 mm were fabricated using standardized  
109 molds and maintained under standard curing conditions. The compressive strength of the concrete

110 was 52.4 MPa and 80.2 MPa respectively at the time of 28 days and T-joint experiment, and the  
111 modulus of elasticity of the concrete was 34,700 N/mm<sup>2</sup>.

## 112 2.2. Test specimens

113 A total of 12 specimens, including 8 T-joints using square CFDST chord, 2 T-joints using square  
114 CFST chord and 2 T-joints using steel SHS chord as counterparts, were tested under monotonically  
115 varied compressive force upon the top of steel SHS brace with the chord concentrically restrained.

116 The parameters used in the experiment are as follows:

- 117 • Chord type: square CFDST with hollow ratio ( $\chi$ ) of 0.3 and 0.5, square CFST and steel SHS;
- 118 • Brace-to-chord width ratio ( $\beta$ ): 0.3 and 0.5; and
- 119 • Concentric compression level of the chord ( $n$ ): 0.04~0.4.

120 The hollow ratio of CFDST chord ( $\chi$ ), brace-to-chord width ratio ( $\beta$ ) and concentric compression  
121 level of the chord ( $n$ ) of the tested specimens are defined as follows,

$$122 \quad \chi = b_i / (b_o - 2t_o) \quad (1)$$

$$123 \quad \beta = b_b / b_o \quad (2)$$

$$124 \quad n = N_0 / N_{cr} \quad (3)$$

125 where,  $N_0$  is a constant concentric compressive load applied to the chord of the T-joints throughout  
126 the whole process of testing; and  $N_{cr}$  is the axial capacity of individual composite and steel chords,  
127 which is calculated using the simplified formulae in [29] and [27], respectively.

128 The specimens are the scaled-down from the actual structural T-joints. The dimensions of the tested  
129 specimens are determined by referring to the previous studies [2, 9-11, 15], and further considering  
130 the limitations of the test site and the capacity of the equipment. The important factors affecting the  
131 static behaviour of the composite T-joints contain material, geometric and load parameters. The  
132 selected steel SHSs with different properties, hollow ratio of CFDST chord ( $\chi$ ), brace-to-chord width  
133 ratio ( $\beta$ ) reflect the material and geometric parameters; whilst the selected concentric compression  
134 level of the chord ( $n$ ) reflects the load parameters. The variation range of the parameters is generally  
135 controlled to represent the T-joint configuration and design method in practice.

136 The configurations of the specimens are illustrated in Fig. 2, where  $b_o$  ( $b_i$ ) and  $t_o$  ( $t_i$ ) are the  
137 overall width and wall thickness of outer (inner) steel SHS in the chord respectively,  $b_b$  and  $t_b$   
138 represent the overall width and the corresponding wall thickness of steel SHS brace, respectively,  $w$   
139 is the weld size, and the remaining dimensions are in mm. The length of the brace and chord in all  
140 specimens is the same, which equals to 400 mm and 1,200 mm, respectively. Table 2 summarizes the  
141 detailed information of the specimens, where  $F_{ue,j}$  is the measured bearing capacity;  $\Delta_{mc,ue}$  and  
142  $\Delta_{tb,ue}$  are the vertical displacement at the chord mid-span and the brace top while achieving  $F_{ue,j}$ ,  
143 respectively; and  $F_{ufe,j}$  is the simulated bearing capacity based on the finite element (FE) model.  
144 Regarding the labels in Table 2, the composite chord and steel SHS chord is respectively denoted by  
145 the capital letters ‘C’ and ‘S’ in the first part, and  $\chi$  of the chord in the composite T-joints follows  
146 the capital letter ‘C’, whilst  $\beta$  and  $n$  are demonstrated in the second and third parts, respectively.

147 Outer and inner SHS of the chords and the SHS braces were all fabricated using cold-formed square  
148 steel tubes. The length of each steel SHS was determined by its length in different T-joint specimens,  
149 and the ends of each steel SHS was further treated as flat before welding. Fillet welds with shielded  
150 metal-arc welding were used to weld the brace and the outer tube of the chord, and the weld sizes ( $w$ ,  
151 as shown in Fig. 2) of the specimens were all greater than  $1.5t_b$ . The 2.5 mm, 3.2 mm and 4.0 mm  
152 electrodes with nominal 0.2% proof stress, tensile strength and elongation of 400 MPa, 480 MPa, and  
153 22%, respectively, were used for welding. The quality of all welds was strictly controlled to meet the  
154 requirements of effective force transfer. At the same time, each chord had two square steel endplates  
155 of 280 mm side length and 20 mm thickness, and each brace had one square steel endplate of 180 mm  
156 side length and 20 mm thickness, as shown in Fig. 2.

### 157 2.3. Test rig and measuring instruments

158 A set of test rig for the T-joints was specially designed, as shown in Fig. 3. One end of the horizontally  
159 placed chord is concentrically restrained through a one-way plate hinge bolted to a fixed reaction  
160 block, whilst the other end is connected to a solid rod capable of moving horizontally. A 2,000 kN  
161 hydraulic jack connecting the solid rod and a fixed reaction block was adopted to apply the constant

162 concentric compressive load ( $N_0$ ), and a 2,000 kN servo actuator connected to the top of brace by a  
163 rigid component was selected to apply the varied compressive force. Moreover, in order to ensure  
164 that the whole T-joint specimen moved in the vertical plane, the possible out-of-plane deformation in  
165 the joint was prevented by two pairs of lateral bracings on both sides of the chord. At the same time,  
166 four bearings were arranged along the height direction of each lateral bracing to eliminate the friction  
167 resistance of the chord wall, hence during the movement of the chord there were always two bearings  
168 in contact with its side walls.

169 Each T-joint specimen was fitted with 4 displacement transducers, among which three were used  
170 to record vertical displacements of key position of the chord and one was used to monitor vertical  
171 displacements at the top of the brace. Furthermore, to examine the representative strain development  
172 of the outer SHS in the chord and the SHS brace, strain gauges were longitudinally and transversely  
173 installed at the mid-span section of the steel tube outside the chord and longitudinally pasted at the  
174 half-height section of the brace, and a total of 14 strain gauges were arranged for each specimen, as  
175 shown in Fig. 3. The forces on the top of the brace and the representative deformations of the chord  
176 and brace were acquired by a data logger.

177 The displacement control loading method with a loading rate of 0.5 mm/min was adopted in the  
178 current tests. The test was terminated when any of the following three conditions were achieved: 1)  
179 the vertical displacement of the chord mid-span section was larger than 1/20 of the chord effective  
180 span, 2) the load resistance decreased to about 60% of the measured peak force, or 3) the obvious  
181 downward displacements of upper flange of the chord were produced at the joint zone.

## 182 **3. Experimental results and discussion**

### 183 **3.1. Loading process and failure patterns**

184 Detailed observation of the experimental process showed that there was no significant change in the  
185 welds, i.e. the quality of the welds could guarantee that the failure of the specimen occurred only in  
186 the chord or brace. The loading process of the T-joints with composite chord was obviously different  
187 from those using steel SHS chord. For the steel T-joints, the concavity damage at the joint zone started

188 from the upper flange of steel SHS chord as well as the bulge appeared at the webs next to the upper  
189 flange increased quickly with increase of the vertical forces until failure happened, and throughout  
190 the loading process the vertical displacements at the lower surface of the chord were limited and there  
191 was no sign of destruction for the brace, [considering that the chord in the steel T-joints pertained to](#)  
192 [the non-compact steel section \[27, 28\]](#). For the composite T-joints with  $\beta$  of 0.3, the interval bulge  
193 damage started at one end or both ends of the brace became more and more obvious with increase of  
194 the forces until failure, and the vertical displacements at the lower surface of the composite chord  
195 were also limited. For the composite T-joints with  $\beta$  of 0.5, either on one side or both sides of the  
196 brace the local buckling of the top flange of the outer chord tube occurred first and then extended to  
197 both webs of the outer chord tube until failure, and the vertical displacements at the lower surface of  
198 the chord continued to increase with increase of the forces.

199 [Fig. 4](#) demonstrates the final failure pattern of all specimens. It is shown that, for the composite T-  
200 joints with  $\beta$  of 0.5, local buckling is observed at upper flange and partial webs close to the upper  
201 flange of the outer SHS in the chord; however, there is no obvious damage to the steel SHS brace.  
202 The difference of buckling position and number of outer tube of the chord may be caused by the  
203 random distribution of initial material defects. For the composite T-joints with  $\beta$  of 0.3, local  
204 buckling at both ends and overall in-plane lateral deformation of steel SHS brace happen and there is  
205 no obvious destruction in the composite chord. However, for the steel T-joints, flange yielding and  
206 web buckling of steel SHS chord are observed under the concentrated loading from the steel SHS  
207 brace [\[27, 28\]](#), and there is no obvious damage to the brace.

208 It was observed from the tests that there were three types of failure pattern for the concrete in  
209 the composite chord, as indicated in [Fig. 5](#). As can be found in [Figs. 5\(a\)](#) and [5\(b\)](#), for the chords  
210 having local buckling in outer steel SHS, crushing of concrete was observed at the buckling positions  
211 together with cracking of concrete in the tension area, and there is a square ring indentation on the  
212 upper surface of the concrete. However, for the chords having local buckling in steel SHS brace, there  
213 is no obvious damage to the concrete ([Fig. 5\(c\)](#)).

214 Similar to the concrete, the inner steel SHS in the square CFDST chords mainly exhibited three  
 215 kinds of failure patterns, as demonstrated in Fig. 6. As can be observed in Fig. 6(a), for the chords  
 216 with  $\chi = 0.5$  and  $\beta = 0.5$ , the combined inward local buckling as well as overall deflection occur  
 217 to the inner steel SHS under the lateral loading and concentric compression due to the decreased local  
 218 stability with a larger  $b_i/t_i$ , and the inward local buckling becomes more serious with increase of  $n$ .  
 219 As can be seen in Fig. 6(b), for the chords with  $\chi = 0.3$  and  $\beta = 0.5$ , only the overall deflection of  
 220 the inner steel SHS is produced irrespective of  $n$  value, as the stability of the inner steel SHS with a  
 221 smaller  $b_i/t_i$  becomes better. It is shown in Fig. 6(c) that, for the composite T-joints with only brace  
 222 destruction ( $\beta = 0.3$ ), the inner steel SHS has no evident damage.

223 The failure patterns of the T-joint specimens observed in the experiments are also summarized  
 224 in Table 2.

### 225 3.2. Force-displacement curves

226 The recorded force ( $F$ ) versus vertical displacement ( $\Delta_{tb}$  and  $\Delta_{mc}$ ) relationship of the composite and  
 227 steel T-joints are illustrated in Fig. 7, where  $F$  is the recorded forces on the top of the steel SHS  
 228 brace, and  $\Delta_{mc}$  and  $\Delta_{tb}$  are the measured vertical displacements at the chord mid-span and at the  
 229 brace top, respectively. The measured bearing capacity ( $F_{ue}$ ) and the vertical displacements  
 230 corresponding to  $F_{ue}$  ( $\Delta_{mc,ue}$  and  $\Delta_{tb,ue}$ ) of all T-joints are presented in Table 2. The recorded peak  
 231 force and the corresponding vertical displacements of the composite T-joints are defined as  $F_{ue}$  and  
 232  $\Delta_{mc,ue}$  ( $\Delta_{tb,ue}$ ), respectively. For the reference steel T-joints, the ultimate deformation limit criterion  
 233 proposed in [30] is employed to determine the bearing capacity. The 1.5 times of force corresponding  
 234 to the deformation of the upper flange of the steel SHS chord ( $\delta_f$ ) reaching  $1\%b_o$  is defined as  $F_{ue}$ ,  
 235 and the corresponding vertical displacements in  $F - \Delta_{tb}(\Delta_{mc})$  curves is thus regarded as  $\Delta_{mc,ue}$   
 236 ( $\Delta_{tb,ue}$ ). The following formulae are adopted to compute  $\delta_f$ :

$$237 \quad \delta_f = \Delta_{tb} - \Delta_{mc} - \delta_b \quad (4-1)$$

$$238 \quad \delta_b = \frac{F \cdot l_b}{A_b \cdot E_{bs}} \quad (4-2)$$

239 where,  $\delta_b$  is the compression deformation of steel SHS brace, and  $E_{bs}$ ,  $A_b$  and  $l_b$  are the

240 modulus of elasticity, cross-sectional area and length of steel SHS brace, respectively.

241 As can be detected from Fig. 7 that, for all specimens  $\Delta_{tb}$  are always greater than  $\Delta_{mc}$  during  
242 the loading process due to the compression deformation of steel SHS brace as well as the partial chord  
243 beneath it. As can be found in Figs. 7(a) and 7(b), when  $\beta = 0.5$ , the composite T-joints having a  
244 bigger  $n$  possess a larger initial slope of  $F - \Delta_{tb}(\Delta_{mc})$  curves and descending slope of  $F -$   
245  $\Delta_{tb}(\Delta_{mc})$  curves after achieving  $F_{ue}$ , and  $\chi$  also has an obvious influence on the descending slope  
246 of  $F - \Delta_{tb}(\Delta_{mc})$  curves. This is probably because the increased constraint induced by the increase  
247 of  $n$  improves the lateral resistant stiffness of the chord, resulting in a larger initial slope; however,  
248 the increase of concentric compression load leads to the increased second-order effect, which  
249 produces a quicker decrease of load after achieving  $F_{ue}$ . Moreover, after reaching  $F_{ue}$ , the inner tube  
250 of square CFDST chord in the composite T-joints having a larger  $\chi$  is apt to buckle owing to a larger  
251  $b_i/t_i$  and thus reducing the support effect of the sandwiched concrete and inner tube to the outer tube.  
252 It is shown in Figs. 7(c) and 7(d) that, the effect of  $\chi$  on the initial and falling slope of  $F - \Delta_{tb}(\Delta_{mc})$   
253 curves of the composite T-joints having  $\beta = 0.3$  as well as  $n = 0.2$  is moderate; however,  $\Delta_{mc}$   
254 quickly decreases after reaching  $F_{ue}$  as the failure only occurs to the brace. Under the same  $\beta$  and  
255  $n$  values, the composite T-joints have a larger initial slope of  $F - \Delta_{tb}(\Delta_{mc})$  curves and descending  
256 slope of  $F - \Delta_{tb}(\Delta_{mc})$  curves after achieving  $F_{ue}$  than the corresponding steel T-joint, because of  
257 the contribution of the inner tube as well as the sandwiched concrete before achieving  $F_{ue}$ , and the  
258 inner tube local buckling and/or the sandwiched concrete crushing after reaching  $F_{ue}$ .

259 It was observed that there were three types of vertical displacement patterns along the effective  
260 span of the chord, as typically shown in Fig. 8, where  $u_c$  is the recorded vertical displacements by  
261 three displacement transducers on the chord,  $L_{ce}$  is the effective span of the chord (1430 mm)  
262 between two hinge supports,  $y$  is the distance from one hinge support, and  $f(= F/F_{ue})$  is the force  
263 ratio and positive and negative values represent the load rising stage before  $F_{ue}$  and the load falling  
264 stage after  $F_{ue}$ , respectively. It is shown that, for the T-joints having chord destruction (e.g. C0.5-0.5-  
265 0.2 or S-0.3-0.2),  $u_c$  continually increase with the variation of  $f$  till the end of the tests as the brace

266 without reaching the bearing capacity can always transmit vertical forces to the chord, and the  
267 distribution of vertical displacements is generally symmetric about the mid-span of the chord;  
268 however,  $u_c$  of steel T-joints are obviously lower than those of the composite counterparts, as the  
269 chord destruction in the steel T-joints mainly appeared in the limited zone of the upper flange of the  
270 steel SHS chord. Moreover, for the composite T-joints having brace destruction (e.g. C0.5-0.3-0.2),  
271  $u_c$  increase with increase of  $f$  before reaching  $F_{ue}$  under the vertical forces transmitted by the  
272 brace, and after that  $u_c$  decrease with decrease of  $f$  due to the elastic recovery of the displacements  
273 of the chord without reaching the bearing capacity, and the distribution of vertical displacements is  
274 not symmetric about the mid-span of the chord.

### 275 3.3. Strain development

276 Fig. 9 demonstrates the force ( $F$ ) versus strain ( $\varepsilon$ ) relationship in the chord of typical specimens at  
277 different measuring points, where  $\varepsilon_{y0}$  is the yield strain of outer chord tube. As can be observed, in  
278 general, the farther the measuring point is from the horizontal centroid axis (e.g. points a, b and d at  
279 the corner and point e at the center of lower flange) the faster the strain develops, and the strain  
280 development at the horizontal centroid axis (point c) are the slowest regardless of the type of the  
281 chord. Meanwhile, the variation of the strains at the upper and lower flange corners is opposite due  
282 to the action of moment produced by the lateral and concentric loads. For the composite T-joints  
283 having chord failure (see Figs. 9(a) and 9(b)), the longitudinal strain corresponding to  $F_{ue}$  at all  
284 measuring points is larger than  $\varepsilon_{y0}$ , which indicates that the property of outer steel SHS can be fully  
285 utilized. However, for the composite T-joints having brace destruction, the strain at each measuring  
286 point changes substantially within the elastic range. Moreover, for the steel T-joints (see Fig. 9(c)),  
287 when achieving peak force the longitudinal strain at all measuring points has significant difference,  
288 i.e. values at the upper part (points a and b on the corner and point c at the horizontal centroid axis)  
289 are larger than  $\varepsilon_{y0}$ , whilst values at the lower corner and flange (points d and e) are much smaller  
290 than  $\varepsilon_{y0}$  as the destruction is concentrated in the upper half of the steel SHS chord (see Fig. 4 (d)).

291 Fig. 10 illustrates the longitudinal strain ( $\varepsilon_L$ ) distribution of the chords under different force ratio

292 ( $f$ ). It is shown that,  $\varepsilon_L$  at the selected points of the chord of all specimens increases with increase  
 293 of ( $f$ ) before reaching  $F_{ue}$ . For the T-joint specimens with the composite chord,  $\varepsilon_L$  is generally  
 294 linear along the height of the section, i.e.  $\varepsilon_L$  at the centroid axis (point c) is always smaller than  $\varepsilon_{y0}$   
 295 and varies within a limited range, and simultaneously, the farther away from the centroid axis, the  
 296 more sufficient the longitudinal strain development, and  $\varepsilon_L$  at point a(b) and e(d) is larger than  $\varepsilon_{y0}$   
 297 when  $n \geq 0.8$ . The concentric compression level of the chord ( $n$ ) mainly affects the initial strain of  
 298 the chords has a moderate effect on the distribution characteristics of  $\varepsilon_L$ , whilst, under the same  $f$   
 299 value, the  $\varepsilon_L$  distribution of CFDST chords is similar to that of CFST chords, which indicates that  
 300 the CFDST chord have similar performance to the corresponding CFST chord due to the presence of  
 301 the inner tube. Moreover, compared with the composite chords, all  $\varepsilon_L$  of steel SHS chords are  
 302 smaller than  $\varepsilon_{y0}$  and the maximum  $\varepsilon_L$  appear at point c due to the web buckling of the side walls  
 303 near the point c.

304 Fig. 11 shows the impact of parameters on  $f - \varepsilon_L$  curve of the steel SHS brace, where  $\varepsilon_L$  is the  
 305 average longitudinal strain recorded by four strain gauges, and  $\varepsilon_{yb,100}$  and  $\varepsilon_{yb,60}$  are the yield  
 306 strain of the steel SHS brace with  $b_b$  of 100 mm and 60 mm, respectively. It is shown that, generally,  
 307 a lower initial slope of  $f - \varepsilon_L$  relationship and a higher longitudinal strain corresponding to  $F_{ue}$   
 308 ( $\varepsilon_{ue}$ ) are produced, with increase of  $\chi$  and  $n$  and decrease of  $\beta$  regardless of the chord type. This  
 309 is mainly attributed to the fact that, the chord flexural stiffness is affected by three experimental  
 310 parameters. Generally,  $\varepsilon_{ue}$  of the composite T-joints having  $\beta = 0.3$  is larger than  $\varepsilon_{by,60}$  (see Fig.  
 311 11(b)) as the failure of specimens is governed by the steel SHS brace.  $\varepsilon_{ue}$  of three composite T-joints  
 312 with  $\beta = 0.5$  (i.e. C0.3-0.5-0.4, C0.5-0.5-0.2 and C0.5-0.5-0.4) is also larger than  $\varepsilon_{by,100}$  (see Fig.  
 313 11(a)) as their  $F_{ue}$  is higher than other specimens with  $\beta = 0.5$ , which have a smaller  $\varepsilon_{ue}$  than  
 314  $\varepsilon_{by,100}$ . In addition,  $\varepsilon_{ue}$  of the steel T-joints is much smaller than their yield strain (see Fig. 11(b))  
 315 as the failure of specimens is controlled by the steel SHS chord.

### 316 3.4. Bearing capacity and the corresponding displacements

317 Fig. 12 demonstrates the effect of parameters on the bearing capacity ( $F_{ue}$ ) of the specimens. As can

318 be observed from Fig. 12(a), while keeping  $\beta$  and  $n$  value constant, the composite T-joints have an  
 319 obviously higher  $F_{ue}$  than the corresponding steel T-joint, and 17.3~18.3 and 12.4~14.7 times  
 320 improvement of  $F_{ue}$  are respectively achieved when  $\beta = 0.3$  and  $\beta=0.5$ . For the composite T-  
 321 joints having  $\beta = 0.3$  and  $n = 0.2$ ,  $F_{ue}$  changes a little with the variation of chord type and  $\chi$ ,  
 322 seeing that the identical steel SHS brace determines the bearing capacity of these specimens and the  
 323 difference may be caused by the variability of different specimen materials. In addition, for the  
 324 composite T-joints having  $\beta = 0.5$  and  $n = 0.2$ , a bigger  $\chi$  of CFDST chord leads to a higher  $F_{ue}$   
 325 due again to the increased section modulus of inner steel SHS, and the specimen with CFDST chord  
 326 leads to 5.7~11.7% larger  $P_{ue}$  than that with CFST chord. It is shown in Fig. 12(b) that, when  $\beta$   
 327 equals to 0.5, the composite T-joints having a larger  $n$  and  $\chi$  generally have a higher  $F_{ue}$ , and in  
 328 the case of  $\chi = 0.3$  ( $\chi = 0.5$ ) specimens with  $n$  of 0.2 and 0.4 have 2.0% (7.4%) and 11.9%  
 329 (14.4%) larger  $F_{ue}$  than the corresponding specimen with  $n$  of 0.04. Similarly, in the case of  $n =$   
 330 0.04~0.4, specimens with  $\chi$  of 0.5 possesses 0.3~5.7% larger  $F_{ue}$  than the corresponding  
 331 specimen with  $\chi$  of 0.3. This is due mainly to the fact that, similar to the common square CFDST  
 332 beam-columns [29], an increased concentric compression restraint can enhance the flexure-shear  
 333 capacity of the composite members when  $n$  is within certain limits. Moreover, the inner tube of  
 334 square CFDST chord with a larger  $\chi$  leads to a larger section modulus and thus a larger moment  
 335 resistance; however, the improvement is limited owing to the second-order effect and difference in  
 336 material property.

337 Fig. 13 demonstrates the change of  $\Delta_{mc,ue}$  and  $\Delta_{tb,ue}$  of all specimens. As can be seen in Fig. 13  
 338 and Table 2,  $\Delta_{tb,ue}$  are larger than  $\Delta_{mc,ue}$  owing to the compression deformation of brace and chord  
 339 beneath the brace. It is shown in Fig. 13(a) that,  $\Delta_{mc,ue}$  and  $\Delta_{tb,ue}$  of the composite T-joints are  
 340 also obviously larger than those of their steel counterparts due again to the contribution of concrete  
 341 and/or inner tube in the chord, and about 1.4~1.9 (2.1~2.2) and 12.2~15.1 (19.2~19.7) times of  
 342  $\Delta_{mc,ue}$  ( $\Delta_{tb,ue}$ ) are respectively achieved when  $\beta = 0.3$  and  $\beta=0.5$ . Moreover, for the composite  
 343 T-joints having  $\beta = 0.3$  as well as  $n = 0.2$ ,  $\Delta_{mc,ue}$  and  $\Delta_{tb,ue}$  of T-joint with CFDST chord

344 having  $\chi$  of 0.3 and 0.5 are larger than those of T-joint with CFDST chord as the flexural stiffness  
345 of the CFDST chord is enhanced owing to the presence of the inner tube, which produces a decreased  
346 chord vertical displacement and an increased the chord restraint to the vertical displacement of brace  
347 for the specimens having brace failure. However, for the composite T-joints having  $\beta = 0.5$  as well  
348 as  $n = 0.2$ , the chord type generally has a moderate effect on the change of  $\Delta_{mc,ue}$  and  $\Delta_{tb,ue}$ . It  
349 can be observed from Fig. 13(b) that,  $\Delta_{mc,ue}$  and  $\Delta_{tb,ue}$  generally decrease with increase of  $n$  and  
350  $\chi$  has a moderate impact on  $\Delta_{mc,ue}$  and  $\Delta_{tb,ue}$  except for the specimens with  $n = 0.04$ . When  $n$   
351 is increased from 0.04 to 0.4 whilst  $\chi$  is maintained the same, 62.8~67% ( $\chi=0.3$ ) and 56.0~60.4%  
352 ( $\chi=0.5$ ) lower  $\Delta_{mc,ue}$  as well as 57.4~59.4% ( $\chi=0.3$ ) and 48.9~55.3% ( $\chi=0.5$ ) lower  $\Delta_{tb,ue}$  are  
353 produced. This is due to the fact that the chord of the composite T-joints having a larger  $n$  reaches  
354 the yield or failure earlier.

#### 355 **4. Finite element (FE) simulation**

356 To capture the static behaviour of the square CFDST chord to steel SHS brace T-joints, a finite  
357 element (FE) model was established based on the nonlinear analysis tool ABAQUS [31].

##### 358 **4.1. Description of the FE model**

359 Four-node reduced integrated shell elements (S4R) were adopted for the modelling of steel SHSs in  
360 the T-joints, and eight-node reduced integrated three-dimensional solid elements (C3D8R) were  
361 employed for the simulating of the concrete in the composite chord and the endplates.

362 Element division of the FE model of the composite T-joints was accomplished by the structured  
363 meshing methods in ABAQUS [31], and based on the balance of computational accuracy and  
364 convergence rate, a reasonable element density was obtained by attempting different mesh sizes.  
365 Moreover, there were dense meshing as well as alignment of mesh nodes in the area where the brace  
366 was connected to the chord, as indicated in Fig. 14.

367 The steel SHSs in the T-joints were modelled by the elastic-plastic model. The measured modulus  
368 of elasticity ( $E_s$ ) and Poisson's ratio ( $\mu_s$ ) in Table 1 were taken as the elastic properties of steel. The  
369 plasticity of steel was represented in the form of true stress and logarithmic plastic strain tabulated

370 data, which was converted using the engineering stress versus strain relationship in [32]. Furthermore,  
371 in order to simplify the calculation reasonably, the endplates of the brace and chord were set to be a  
372 type of rigid material with modulus of elasticity and Poisson's ratio of  $1.0 \times 10^{12}$  N/mm<sup>2</sup> and 0.00001,  
373 respectively.

374 The concrete in the composite chord was simulated by the concrete damage plasticity (CDP) model.  
375 The modulus of elasticity of concrete ( $E_c$ ) was equal to  $4730\sqrt{f'_c}$  [33] together with the Poisson's  
376 ratio ( $\mu_c$ ) of 0.2 [34] in the elastic stage, where  $f'_c$  is the cylindrical compressive strength. Within  
377 the plastic stage, the concrete subject to compression was depicted by the stress-strain relationship in  
378 [35] by using the confinement factor  $\xi$  ( $= \alpha_n \cdot f_{yo}/f_{ck}$ ) as main variable, where  $\alpha_n$  is the nominal  
379 steel ratio of a composite chord,  $f_{yo}$  is the yield strength of outer steel SHS, and  $f_{ck}$  is the  
380 characteristic compressive strength of concrete [5, 29]. The tension-stiffening effect of concrete  
381 subject to tension was described by the fracture energy cracking model, in which the peak tensile  
382 stress equals to  $0.1f'_c$  and the fracture energy ( $G_f$ ) suggested in [34] was employed. To characterize  
383 the concrete damage induced by the deformation in the plastic phase, the evolutions of compression  
384 (tension) damage variable were computed using the equations suggested in [36].

385 The FE model of the composite T-joints had five independent parts, i.e. two steel tubes of the chord,  
386 steel SHS of the brace, the concrete in the chord, and endplates of the chord and brace. The  
387 interactions between two steel tubes and the concrete in the chord was captured by the surface-to-  
388 surface contacts, which were also used between three parts of the chord and the corresponding  
389 endplates. The normal and tangential direction of the interface were respectively defined as a hard  
390 contact and a Coulomb friction model with the friction coefficients of 0.6 [37]. In addition, the  
391 interfacial performance between the brace and the outer steel tube of the chord or its endplate was  
392 modelled by the 'TIE' contact.

393 Fig. 14 illustrates the boundary conditions used in the FE model. The plate hinges in the  
394 experiments (shown as in Fig. 3) were simulated by defining two reference points (Pr1 and Pr2),  
395 which were coaxial with the centroid of chord section, and there was a distance of 115 mm between

396 the chord endplate and the corresponding reference point. The sphere hinge on the actuator (also  
397 shown as in Fig. 3) was modelled by the third reference point (Pr3), which was located in the plane  
398 of symmetry along Y axis, and there was a distance of 600 mm between the brace endplate and the  
399 Pr3. The restricted translations included: three directions of Pr1 ( $U_X=U_Y=U_Z=0$ ), two directions of  
400 Pr2 ( $U_X=U_Z=0$ ) and one direction of Pr3 ( $U_X=0$ ), and all three reference points had the restricted  
401 rotations in two directions ( $U_R_Y=U_R_Z=0$ ). In addition, to reproduce the imperfections during the  
402 processing and testing of the specimens, an initial eccentricity ( $e_i$ ) of  $L_{be}/1000$  with respect to the  
403 symmetry axis along X-axis was applied to the Pr3, as shown in Fig. 14, where  $L_{be}$  is the brace  
404 effective length. Firstly, the constant axial compressive load ( $N_0$ ) was applied to the Pr2, and then the  
405 displacements along the Z-axis were acted on the Pr3 until the modelling ends.

406 To further investigate the effect of welds, another FE model with the welds between the brace and  
407 the outer tube of the chord was also built. In this model, the welds were defined as an elastic material  
408 with size ( $w$ ), yield strength, modulus of elasticity and Poisson's ratio of  $1.5t_b$ , 400 MPa,  $2.06 \times 10^5$   
409 N/mm<sup>2</sup> and 0.3, respectively, and were simulated by C3D8R elements. The 'TIE' contact was adopted  
410 to model the interface between the welds and the brace or the outer tube of the chord. The comparison  
411 of FE simulation results of the T-joint specimens with and without welds is demonstrated in Fig. 15.  
412 It can be observed that the influence of the welds on the static performance of the T-joint specimens  
413 is small, and thus the welds between the brace and the outer tube of the chord were not considered in  
414 the FE model.

#### 415 **4.2. Verifications of the FE modelling**

416 The FE modelling results are evaluated against the experimental observations. Fig. 16 illustrates the  
417 simulated failure patterns. The contrast between Fig. 16(a) and Fig. 4 shows that, the simulated  
418 buckling pattern and positions of top flange and webs of outer chord tube and overall deflection shape  
419 of chord in the composite T-joints having  $\beta$  of 0.5, the local buckling of brace in the composite T-  
420 joints having  $\beta$  of 0.3, and the yielding of top flange and buckling of webs of chord in the steel T-  
421 joints generally accord well with the experimental phenomena. The contrast between Fig. 16(b) and

422 Fig. 5 indicates that, for the infilled concrete, the modelled distribution of main plastic strains (PE)  
423 and the position where the maximum PE occurs generally accord well with the observed pattern and  
424 location of compressive crushing and tensile cracking in the tests. As can be seen from Fig. 16(c) and  
425 Fig. 6, a relatively good correlation between the computed overall deflection and/or local buckling of  
426 inner tube of the CFDST chord and the experimental results. The simulated failure patterns are also  
427 given in Table 2.

428 Fig. 17 demonstrates the contrast between the numerical and measured  $F - \Delta_{tb}$  curves, where the  
429 experimental data of steel SHS chord to square CFDST X-joints with the chord end free have also  
430 been adopted to increase the reliability of the FE model. It is shown that, the computed developing  
431 trend of  $F - \Delta_{tb}$  curves are generally accord well with the measured ones. However, the numerical  
432  $F - \Delta_{tb}$  curves of the composite T-joints have an apparently higher initial slope than the  
433 experimntal results. There are three main reasons for this: 1) the FE model cannot reproduce the  
434 imperceptible gaps between different parts, 2) the joints with a larger size have more defects than the  
435 specimens having a smaller size for conducting a material test, and 3) it is impossible to invariably  
436 maintain the ideal concentric compression of the chord in the experiment. The contrast between the  
437 numerical and measured  $F - \varepsilon_L(\varepsilon_T)$  relationship is indicated in Fig. 18, where  $\varepsilon_L$  and  $\varepsilon_T$   
438 respectively stand for longitudinal and transverse strain. It is shown that the slope of the numerical  
439  $F - \varepsilon_L(\varepsilon_T)$  curves is generally close to that of the experimental results, and compared to the  
440 numerical  $F - \Delta_{tb}$  curves, the  $F - \varepsilon_L(\varepsilon_T)$  curves have a better computational initial slope. This  
441 can be explained that, the strain measurement is carried out in a very small range, while the  
442 displacement measurement is relative to the chord span, and the former is less affected by the  
443 aforementioned three types of defects. The computed bearing capacity of the T-joints by FE model  
444 ( $F_{ufe,j}$ ) and  $F_{ufe,j}/F_{ue,j}$  values are given in Table 2, and Fig. 19 shows the contrast between  $F_{ufe,j}$   
445 and  $F_{ue,j}$ . The comparison demonstrates that, the computational results are generally reasonable,  
446 seeing that  $F_{ufe,j}/F_{ue,j}$  values have a mean and standard deviation (SD) of 0.950 and 0.063,  
447 respectively.

448 Although there is a certain discrepancy in the initial slope between the computed and recorded  
449 force versus displacement (strain) relationship, the simulated failure patterns, bearing capacity and  
450 displacement (strain) evolution generally exhibit a good correlation with the experimental  
451 observations. Therefore, the FE model can be regarded as a good predictor for reproducing the  
452 performance of the square CFDST chord to steel SHS brace T-joints.

## 453 **5. Simplified formulae for computing the bearing capacity**

454 The above test and FE modelling results indicate that, the performance of square CFDST chord to  
455 steel SHS brace T-joints with  $\beta$  of 0.3 and 0.5 is mainly dominated by that of individual steel SHS  
456 brace and square CFDST chord, respectively, and thus the bearing capacity of the composite T-joints  
457 should be associated with that of their chord and brace. To find out the connection between the bearing  
458 capacity of individual components and that of the composite T-joints, FE simulations on the behaviour  
459 of the individual brace and chord under static loading were further performed. Figs. 20(a) and (b)  
460 shows the FE model of the individual components. The individual steel SHS brace had the same  
461 conditions as that in the model of joints with the vertical displacements applied from the top and an  
462 initial eccentricity of  $L_{be}/1000$ , except that all degree-of-freedom of the bottom cross-section  
463 connected with the chord in the joint were restricted. It should be pointed out that, in order to simplify  
464 the FE modelling and facilitate the future design, the flexural deformation of the top wall of the chord  
465 connected with the bottom cross-section of steel SHS brace was temporarily ignored for the individual  
466 brace model, as the flexural deformation of the top wall of the chord is usually very small in reality,  
467 unless significant lateral displacement of the brace is presented after reaching the bearing capacity of  
468 the joint. Moreover, the individual square CFDST chord also possessed the same conditions as that  
469 in the model of joints, except that the compression from the brace was replaced by a compressive  
470 square area with the same width as the outer width of the brace, and the lateral displacements were  
471 applied by a reference point (Pr3) coupled with the loading area. Figs. 20(c) and (d) demonstrate the  
472 simulated failure pattern of typical individual components. It can be observed from the comparison  
473 between Figs. 20(c) and (d) and Figs. 4 and 16(a) that, the individual brace and chord have the same

474 failure patterns as those of the corresponding components in the composite T-joints with  $\beta = 0.3$   
 475 and  $\beta = 0.5$ . The contrast between the computed bearing capacity of the composite T-joints together  
 476 with the individual components and  $F_{ue}$  is summarised in Table 3, in which  $F_{ufe,b}$  and  $F_{ufe,c}$   
 477 respectively represent the bearing capacity of individual brace and chord computed by the FE model,  
 478 and  $F_{ufe,bc}$  equals to the minimum of  $F_{ufe,b}$  and  $F_{ufe,c}$ . The results indicate that,  $F_{ufe,bc}$  and  $F_{ufe,j}$   
 479 are very close and have the similar statistical indexes compared to  $F_{ue,j}$ , which means that the bearing  
 480 capacity of the composite T-joints is determined by the minimum bearing capacity of their individual  
 481 components.

482 The FE model was employed for extensive parametric analysis to further confirm the above  
 483 viewpoint, and the calculating parameters included:  $b_o = 400$  mm,  $\beta = 0.1\sim 0.9$ ,  $\chi = 0\sim 0.75$ ,  
 484  $n = 0\sim 0.75$ ,  $\alpha_n = 0.05\sim 0.2$ , slenderness ratio of the square CFDST chord  $\lambda = 10\sim 80$ , yield  
 485 strength of steel SHS brace (inner and outer tube in the chord)  $f_{yb}(f_{yi}, f_{yo}) = 235\sim 460$  MPa,  
 486  $f'_c = 25\sim 75$  MPa,  $b_i/t_i = 20\sim 80$ ,  $b_b/t_b = 10\sim 30$ , and length to width ratio of brace  
 487  $l_b/b_b = 5\sim 20$ . A total of 239 joint models covering the parameter range were simulated.

488 To facilitate analysis, a bearing capacity factor ( $K_1$ ) is defined as follows:

$$489 \quad K_1 = \frac{F_{ufe,j}}{\min\{F_{ufe,b}, F_{ufe,c}\}} \quad (5)$$

490 The variation of  $K_1$  with  $F_{ufe,b}/F_{ufe,c}$  values is demonstrated in Fig. 21. The results indicate that  
 491  $K_1$  values are from 0.962 to 1.043, with mean and SD of 0.996 and 0.011, respectively. Particularly,  
 492  $K_1$  values of the joint having failure of brace (i.e.  $K_1 = F_{ufe,j}/F_{ufe,b}$ ) are from 0.986 to 1.043, with  
 493 mean and SD of 1.008 and 0.010, respectively. The comparison results in Table 3 and Fig. 21  
 494 demonstrate that, within the range of the experimental and FE simulation parameters of this study, it  
 495 is generally feasible to assume that the bottom cross-section of the steel SHS brace connected with  
 496 the chord in the joint is under an ideal rigid restriction while calculating the bearing capacity of  
 497 individual steel SHS brace.

498 Currently, simplified equations for the bearing capacity of the steel SHS members can be found in

499 the design codes, e.g. EN 1993-1-1 [27]; however, there is no specific method for square CFDST  
500 members subjected to the combined concentric compression as well as lateral local compression  
501 acting on the mid-span (see Fig. 20(b)). Therefore, it is necessary to establish the connection between  
502 the square CFDST members subjected to the combined concentric compression as well as lateral local  
503 compression acting on the mid-span and those subject to the combination of compression, flexure and  
504 shear, because the formulae for the bearing capacity prediction of the latter can be obtained in the  
505 literature [29, 38]. Fig. 22 demonstrates the comparison of square CFDST members under different  
506 loading conditions, where CFS represents the combination of compression, flexure and shear. In the  
507 FE modelling, the square CFDST member under CFS had the same conditions as those of the  
508 individual square CFDST chord in the T-joint, except that there is no square loading area and the mid-  
509 span lateral displacements were applied by a rigid plate fixed with the cross-section. The results  
510 indicate that, square CFDST members subject to CFS have different force-displacement curves (Fig.  
511 22(a)) and failure patterns (Fig. 22(b)) compared to those under the combined concentric compression  
512 and lateral local compression at the mid-span. The individual chords with a bigger brace-to-chord  
513 width ratio ( $\beta$ ) have a closer performance to the corresponding members under CFS, and the  
514 individual chords with a very small  $\beta$  (e.g.  $\beta = 0.1$ ) are more prone to joint zone failure.

515 Another bearing capacity factor ( $K_0$ ) is thus defined to measure the difference in bearing capacity  
516 of square CFDST members under the above two loading conditions, and the equation is:

$$517 \quad K_0 = \frac{F_{ufe,c}}{F_{ufe,0}} \quad (6)$$

518 where,  $F_{ufe,0}$  is the numerical lateral bearing capacity of square CFDST members under the  
519 combination of compression, flexure and shear.

520 It is observed from the simulated results that, the bearing capacity factor ( $K_0$ ) is mainly affected by  
521  $\beta$ ,  $\chi$  and  $\lambda$  no matter what pattern of failure occurs to the square CFDST chord, and other  
522 parameters only have slight influence on  $K_0$ . Fig. 23 demonstrates the influence of  $\beta$ ,  $\chi$  and  $\lambda$  on  
523  $K_0$ . According to a large number of parametric analysis results, the simplified formulae for  $K_0$  is  
524 obtained based on regression analysis method:

525 
$$K_0 = [0.12 \ln(\beta) + 1.08] \cdot (p_1 \cdot \chi^2 + p_2 \cdot \chi + p_3) \cdot (q_1 + q_2 \cdot e^{-0.025\lambda/q_3}) \quad (7)$$

526 where,  $p_{i(i=1,2,3)}$  and  $q_{i(i=1,2,3)}$  are calculating parameters,  $p_1 = -3.43\beta^2 + 5.34\beta - 2.02$ ,  
 527  $p_2 = 1.33\beta^2 - 1.96\beta + 0.67$ ,  $p_3 = -0.23\beta^2 - 0.26\beta + 1.04$ ,  $q_1 = 1.59\beta^2 - 1.28\beta + 1.51$ ,  
 528  $q_2 = 5.34\beta^2 - 1.37\beta - 1.7$ , and  $q_3 = 1.15\beta^2 - 1.46\beta + 0.56$ .

529 The calculated  $K_0$  using Eq. (7) are also presented in Fig. 23. It can be seen that, Eq. (7) is suitable  
 530 for accurately calculating the bearing capacity coefficient  $K_0$ .

531 According to Eqs. (5) to (7), the bearing capacity of the square CFDST chord to steel SHS brace  
 532 T-joints ( $F_{u,j}$ ) can therefore be derived as follows:

533 
$$F_{u,j} = \begin{cases} F_{u,b} & (\theta < 1) \\ K_0 \cdot F_{u,0} & (\theta \geq 1) \end{cases} \quad (8)$$

534 where,  $F_{u,b}$  is the stability bearing capacity of individual SHS brace under concentric compression  
 535 based on the formulae in [27];  $F_{u,0}$  is the lateral bearing capacity of individual square CFDST chord  
 536 subject to CFS; and the parameter  $\theta (= F_{u,b}/(K_0 \cdot F_{u,0}))$  should be calculated according to the  
 537 standard value of material strength in the design. In this study, the design formulae in [29, 38] are  
 538 adopted to compute  $F_{u,0}$ , and the detailed formulae are as follows:

539 
$$\begin{cases} \left( \frac{1}{\varphi} \cdot \frac{N_0}{N_u} + \frac{a}{d} \cdot \frac{M}{M_u} \right)^{2.4} + \left( \frac{V}{V_u} \right)^2 = 1 & \left( \frac{N_0}{N_u} \geq 2\varphi^3 \cdot \eta_0 \cdot \sqrt{1 - \left( \frac{V}{V_u} \right)^2} \right) \\ \left( -b \cdot \frac{N_0^2}{N_u^2} - c \cdot \frac{N_0}{N_u} + \frac{1}{d} \cdot \frac{M}{M_u} \right)^{2.4} + \left( \frac{V}{V_u} \right)^2 = 1 & \left( \frac{N_0}{N_u} < 2\varphi^3 \cdot \eta_0 \cdot \sqrt{1 - \left( \frac{V}{V_u} \right)^2} \right) \end{cases} \quad (9)$$

540 where,  $\varphi$  is the stability coefficient;  $N_u$ ,  $M_u$  and  $V_u$  are the sectional strength subject to  
 541 concentric compression, flexure and shear, respectively [29, 38];  $a$ ,  $b$ ,  $c$  and  $\eta_0$  are the  
 542 intermediate variables;  $d$  is the factor considering the second-order effect;  $M$  is the external  
 543 moment calculated by  $F_{u,0} \cdot L_{ce}/4$ ; and  $V$  is the external shear force equal to  $F_{u,0}/2$ .

544 Fig. 24 demonstrates the contrast between the simplified bearing capacities ( $F_{us,j}$ ) using Eq. (8)  
 545 and the numerical results ( $F_{ufe,j}$ ), and the mean and SD of  $F_{us,j}/F_{ufe,j}$  values are equal to 0.962 and  
 546 0.052, respectively. The simplified bearing capacities ( $F_{us,j}$ ) using Eq. (8) are further compared to the  
 547 experimental results ( $F_{ue,j}$ ) in Fig. 25. The mean and SD of  $F_{us,j}/F_{ue,j}$  values are equal to 0.859 and

548 0.072, respectively. It can be observed from the above comparison that the suggested simplified  
549 equations are capable of effectively computing the bearing capacity of the square CFDST chord to  
550 steel SHS brace T-joints. It should be noted that, while using Eq. (8) in design of composite T-joints,  
551 the chord concrete of higher strength class should be used in line with higher grade steel tube [5] to  
552 ensure the composite actions between steel tubes and the sandwiched concrete, and the buckling  
553 resistance of the SHS brace should match the lateral capacity of the CFDST chords under CFS.

## 554 **6. Conclusions**

555 The performance of square CFDST chord to steel SHS brace T-joints under static loading are  
556 experimentally and numerically investigated in this study, and the main conclusions are as follows:

557 1) Compared to bare steel SHS T-joints, the composite T-joints with square CFDST/CFST chord  
558 have better static performance and significantly different failure patterns.

559 2) For the composite T-joints having destruction of chord (i.e.  $\beta = 0.5$ ), the local buckling of top  
560 flange and webs of outer chord tube and inward local buckling and overall deflection of inner chord  
561 tube, together with crushing under compression and cracking under tension of the concrete between  
562 the tubes, are observed. For the composite T-joints having destruction of brace (i.e.  $\beta = 0.3$ ), the  
563 local buckling near the top and bottom section of brace appears.

564 3) For the composite T-joints having destruction of chord, a bigger  $n$  results in a higher initial  
565 slope of force-displacement (strain) relationship and a larger bearing capacity ( $F_{ue}$ ), and generally  
566 specimens with  $n$  of 0.2 and 0.4 have 2.0-7.4% and 11.9-14.4% larger  $F_{ue}$  than those with  $n$  of  
567 0.04. In addition, a larger  $n$  and  $\chi$  leads to a higher descending slope of load versus deformation  
568 curves after achieving  $F_{ue}$ . For the composite T-joints having destruction of brace, the chord type has  
569 no evident impact on the initial and falling slope of force-displacement (strain) relationship.

570 4) Under the same  $\beta$  and  $n$  values, the composite T-joints possess a significantly larger initial  
571 slope of the force-displacement (strain) relationship and  $F_{ue}$ , and a quite smaller falling slope of  
572 force-displacement (strain) relationship after achieving  $F_{ue}$  than the corresponding steel T-joints.  
573 Generally, the composite T-joints having  $\beta = 0.3$  and  $\beta=0.5$  respectively result in 17.3~18.3 and

574 12.4~14.7 times improvement of  $F_{ue}$  over the corresponding steel T-joints.

575 5) The FE model is validated by the experimental results. The simplified formulae for computing  
576 the bearing capacity of the composite T-joints was developed, and the precision of the suggested  
577 equations is validated by numerical and experimental results. Therefore, the simplified formulae can  
578 be used for future design.

### 579 **Declaration of Competing Interest**

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

### 582 **Acknowledgements**

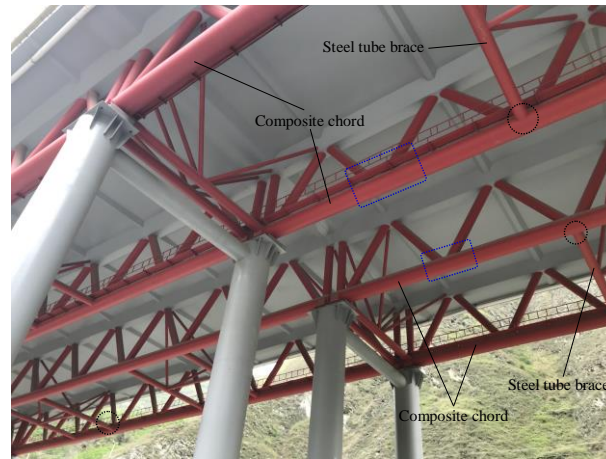
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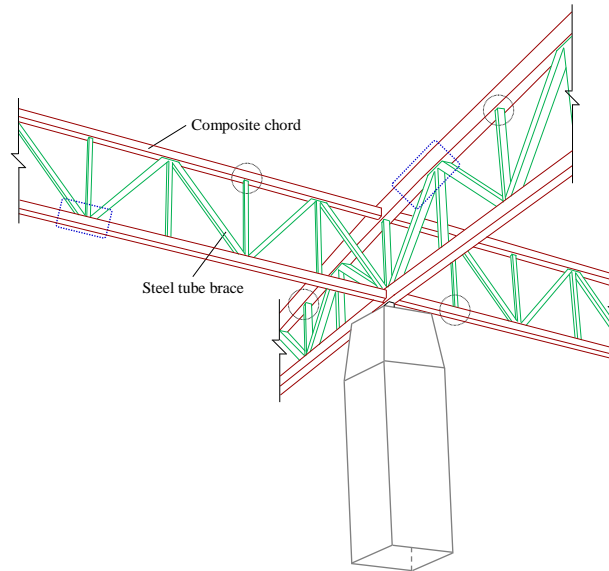
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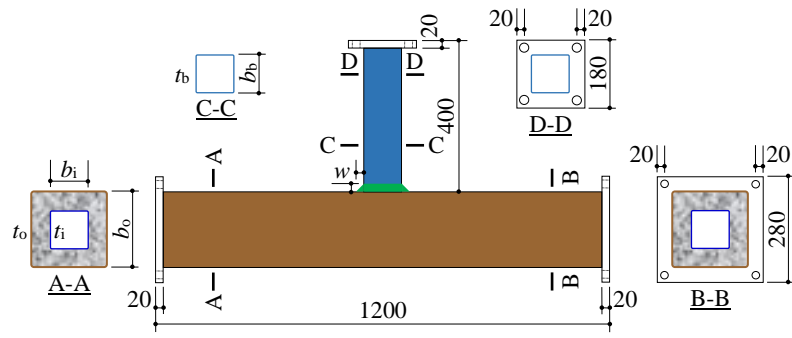
**Figures:**

(a) On-sight photo (with pre-stressed chords)

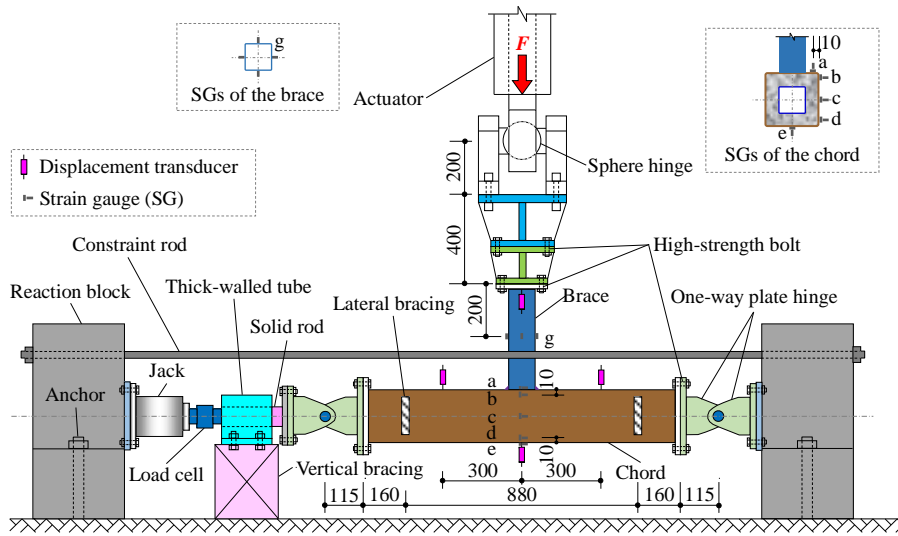


(b) Schematic view

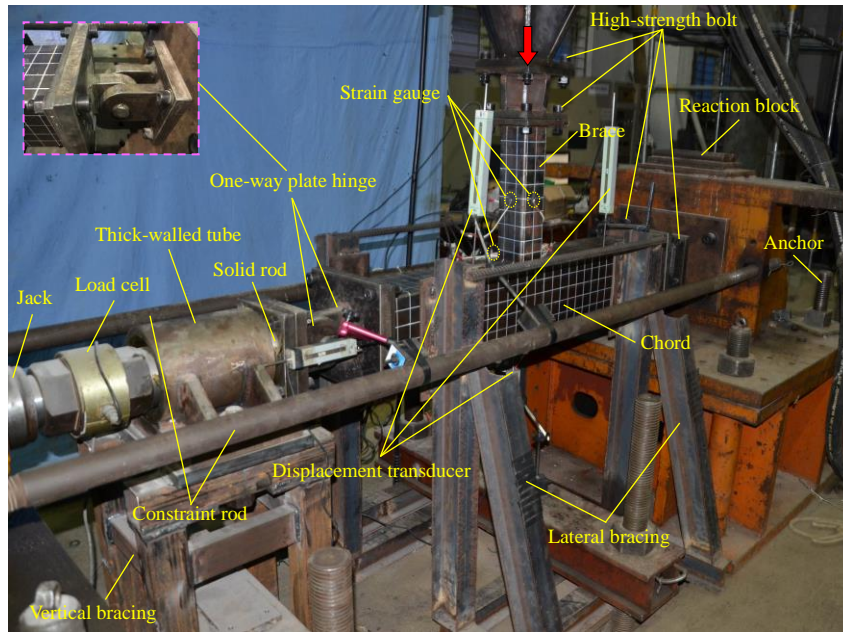
**Fig. 1.** Typical on-sight photo and schematic view of composite tubular joints.



**Fig. 2.** Configurations of the specimens

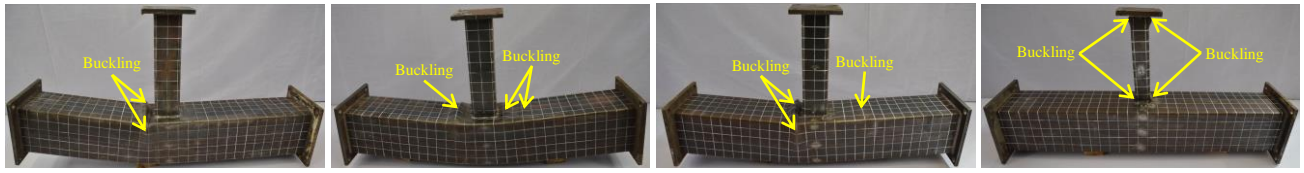


(a) Schematic view (unit: mm)



(b) Actual situation

**Fig. 3.** Test rig and measuring instruments



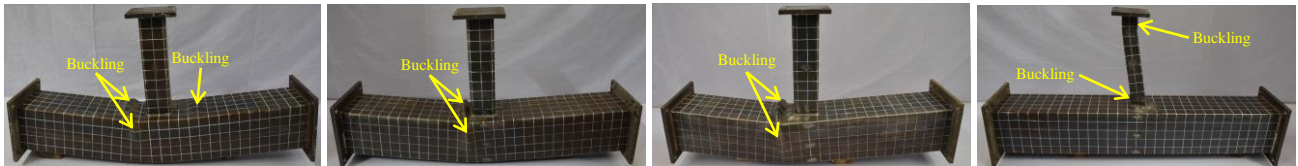
(1) C0.3-0.5-0.04

(2) C0.3-0.5-0.2

(3) C0.3-0.5-0.4

(4) C0.3-0.3-0.2

(a) CFDST chord ( $\chi=0.3$ )



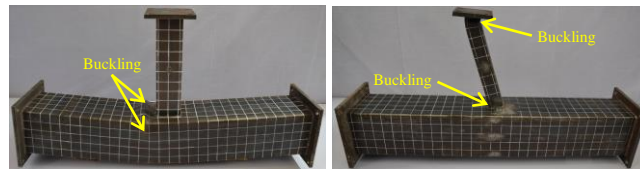
(1) C0.5-0.5-0.04

(2) C0.5-0.5-0.2

(3) C0.5-0.5-0.4

(4) C0.5-0.3-0.02

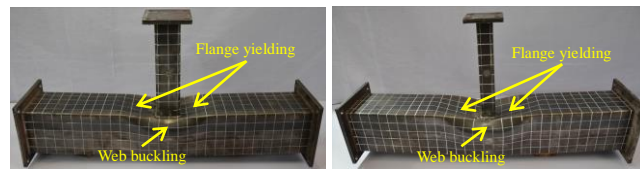
(b) CFDST chord ( $\chi=0.5$ )



(1) C-0.5-0.2

(2) C-0.3-0.2

(c) CFST chord

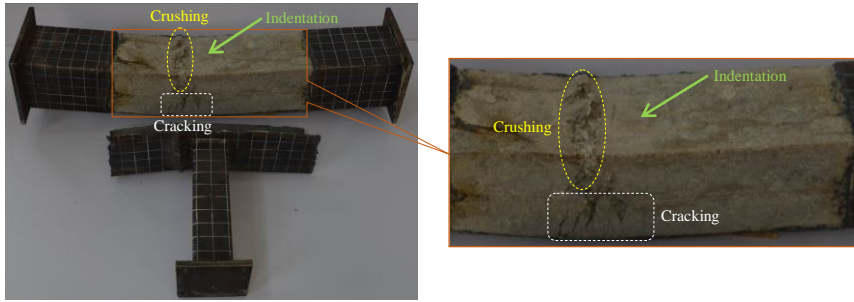


(1) S-0.5-0.2

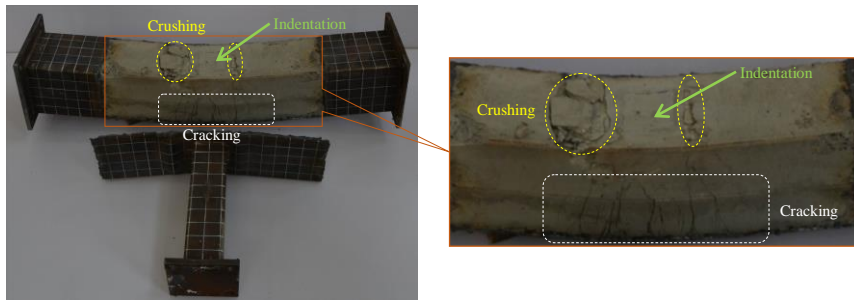
(2) S-0.3-0.2

(d) Steel SHS chord

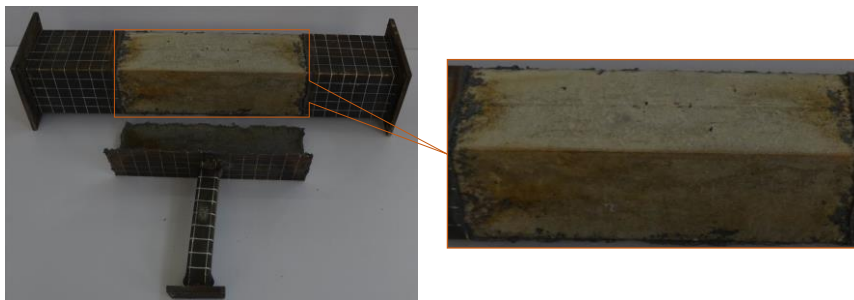
**Fig. 4.** Failure pattern of the specimens



(a) C0.3-0.5-0.04

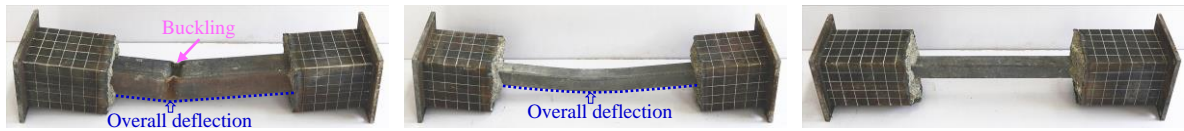


(b) C0.3-0.5-0.2



(c) C0.3-0.3-0.2

**Fig. 5.** Representative failure pattern of the concrete in the composite chord

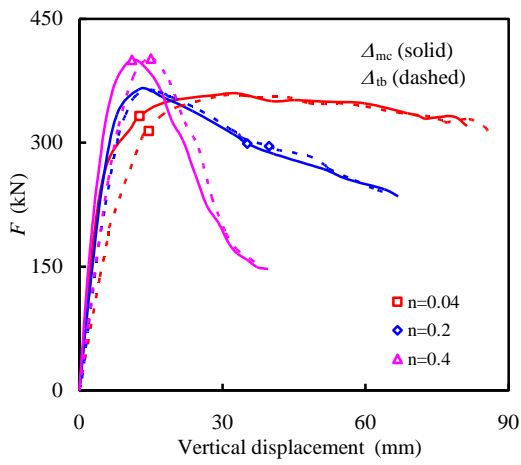


(a)  $\chi=0.5, \beta=0.5$

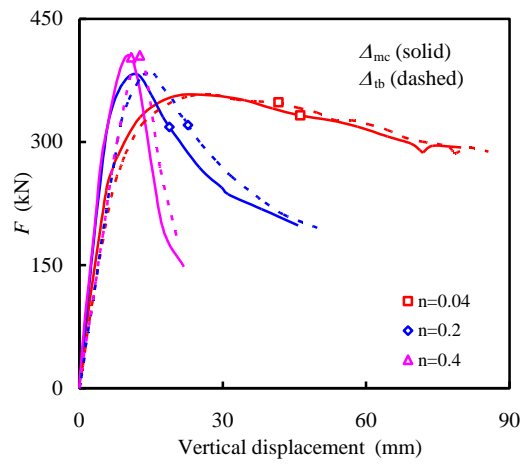
(b)  $\chi=0.3, \beta=0.5$

(c)  $\chi=0.3, \beta=0.3$

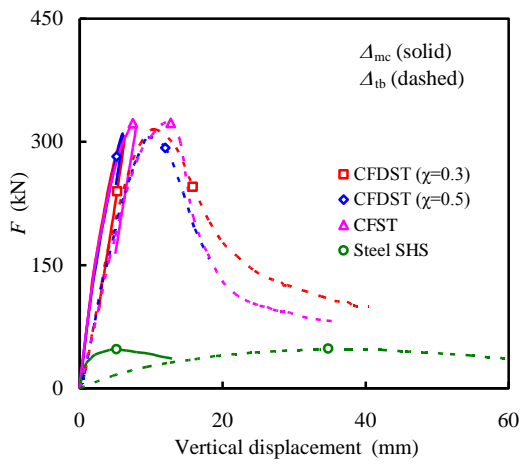
**Fig. 6.** Typical failure pattern of inner steel SHS in the CFDST chord



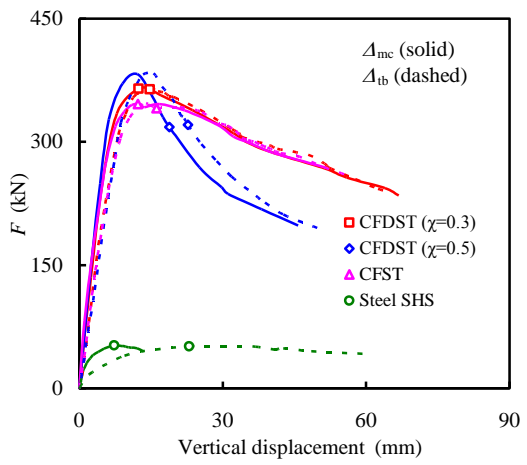
(a) CFDST chord ( $\chi=0.3, \beta=0.5$ )



(b) CFDST chord ( $\chi=0.5, \beta=0.5$ )

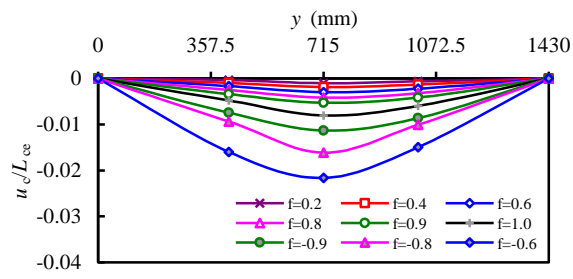


(c) Varied chords ( $\beta=0.3, n=0.2$ )

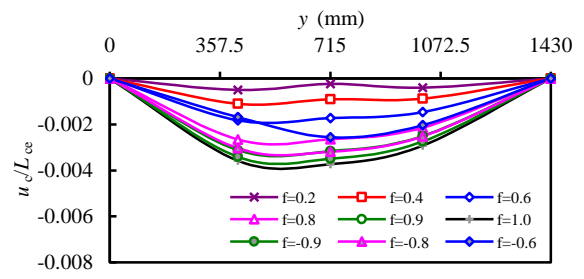


(d) Varied chords ( $\beta=0.5, n=0.2$ )

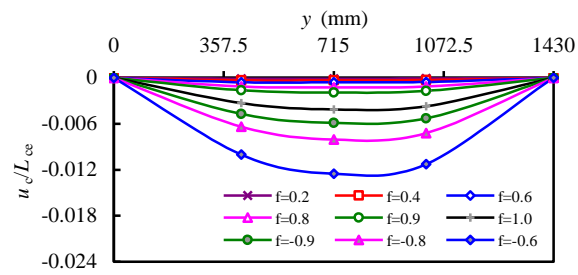
**Fig. 7.** Force versus vertical displacement relationship of the specimens



(a) C0.5-0.5-0.2

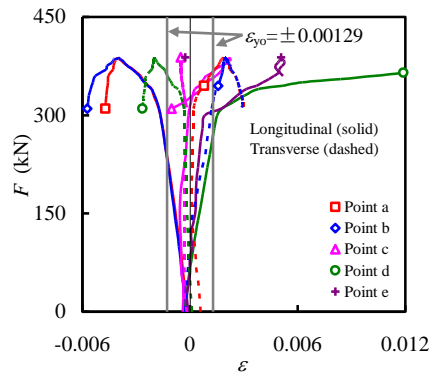


(b) C0.5-0.3-0.2

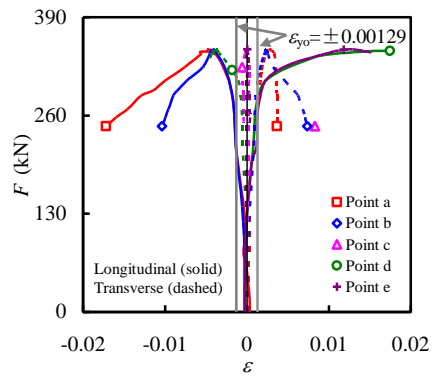


(c) S-0.3-0.2

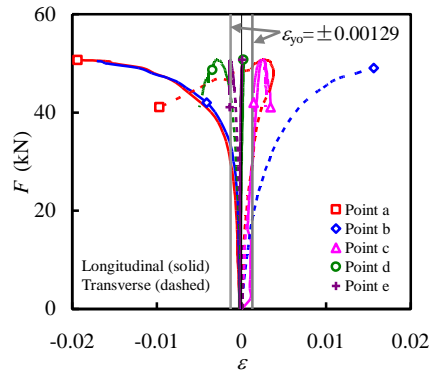
**Fig. 8.** Typical distribution pattern of vertical displacements along the span of the chord



(a) C0.5-0.5-0.2

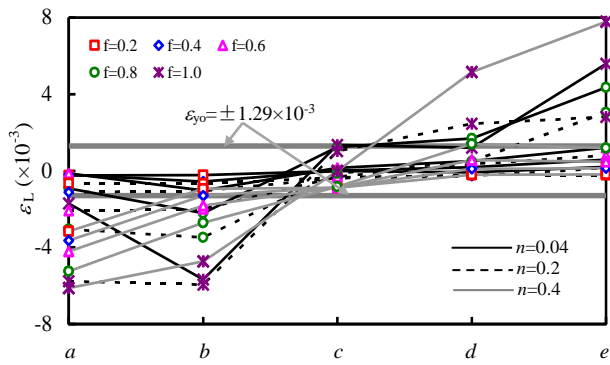


(b) C-0.5-0.2

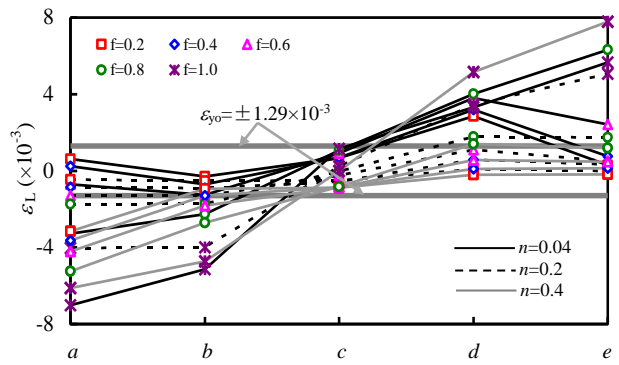


(c) S-0.5-0.2

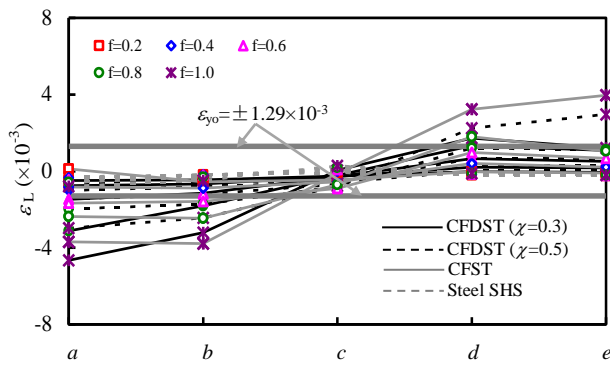
**Fig. 9.**  $F$ - $\epsilon$  relationship in the chord of typical specimens



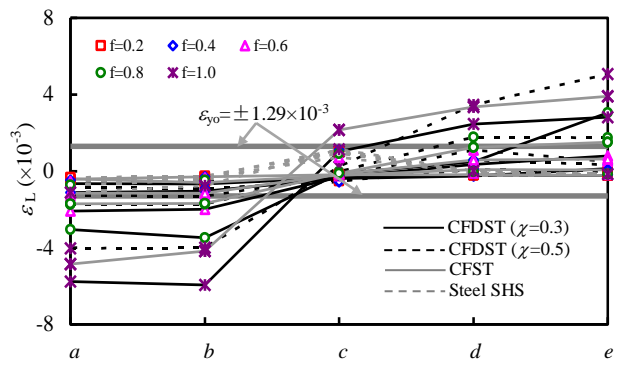
(a) CFDST chord ( $\chi=0.3, \beta=0.5$ )



(b) CFDST chord ( $\chi=0.5, \beta=0.5$ )

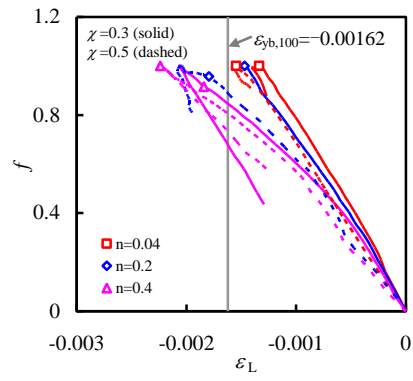


(c) Varied chords ( $\beta=0.3, n=0.2$ )

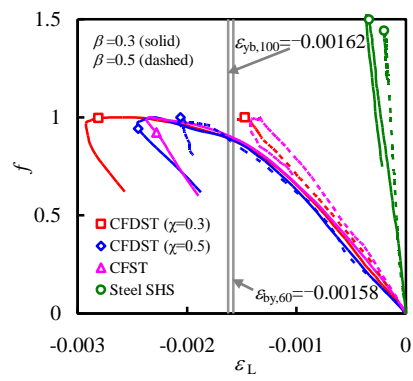


(d) Varied chords ( $\beta=0.5, n=0.2$ )

**Fig. 10.** Longitudinal strain ( $\epsilon_L$ ) distribution of the chords under different force ratio ( $f$ )

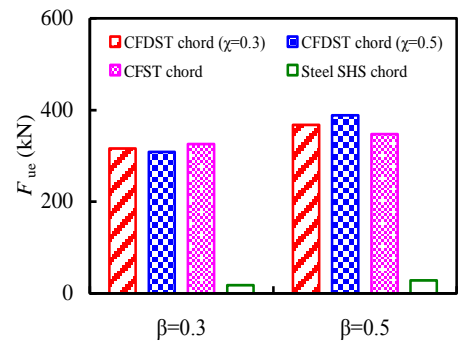


(a) CFDST chord ( $\beta=0.5$ )

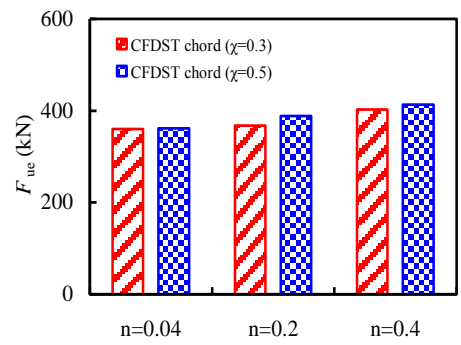


(b) Varied chords ( $n=0.2$ )

**Fig. 11.** Impact of parameters on  $f$ - $\varepsilon_L$  curves of the steel SHS brace

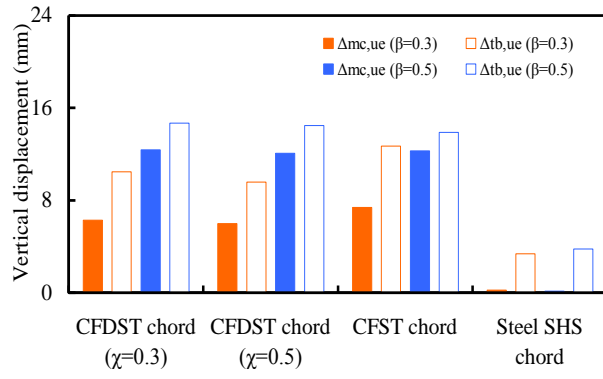


(a) Variation of chord type and  $\beta$  ( $n=0.2$ )

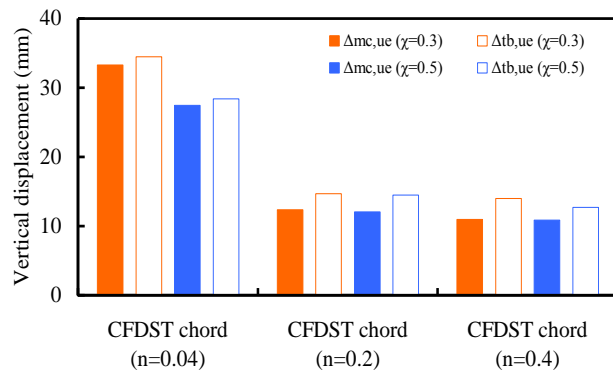


(b) Variation of  $\chi$  and  $n$  ( $\beta=0.5$ )

**Fig. 12.** Effect of parameters on  $F_{ue}$

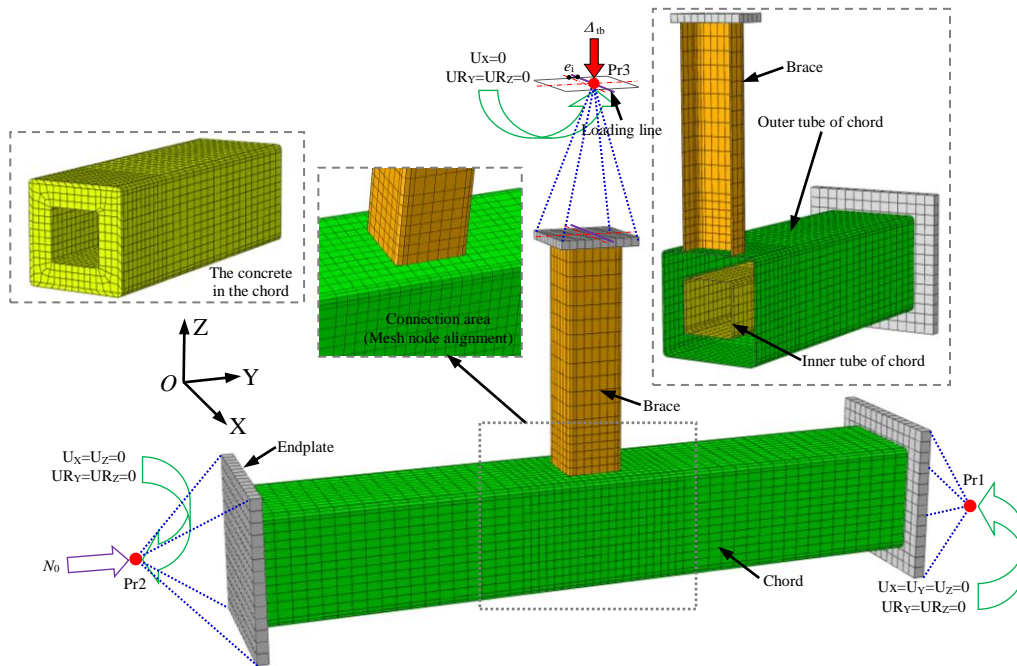


(a) Variation of chord type and  $\beta$  ( $n=0.2$ )

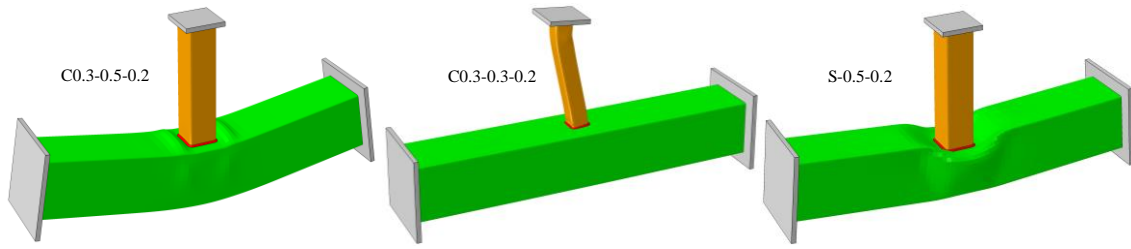


(b) Variation of  $\chi$  and  $n$  ( $\beta=0.5$ )

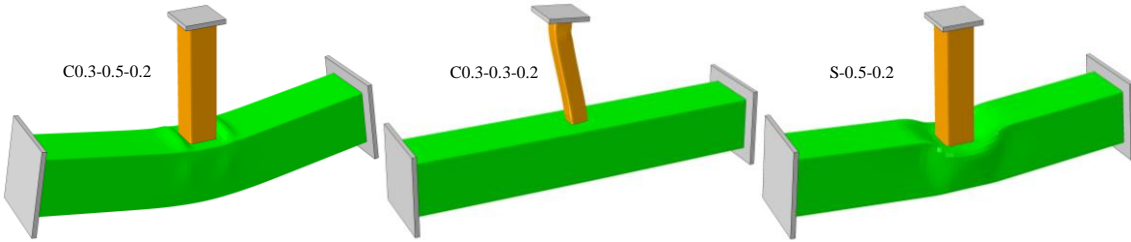
**Fig. 13.** Change of  $\Delta_{mc,ue}$  and  $\Delta_{tb,ue}$



**Fig. 14.** The FE model with meshing and boundary conditions

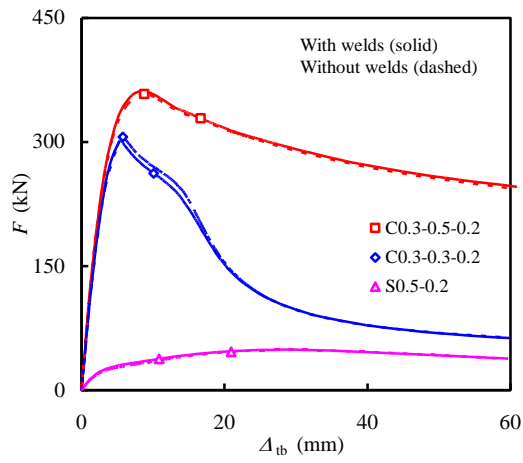


(1) Results with welds



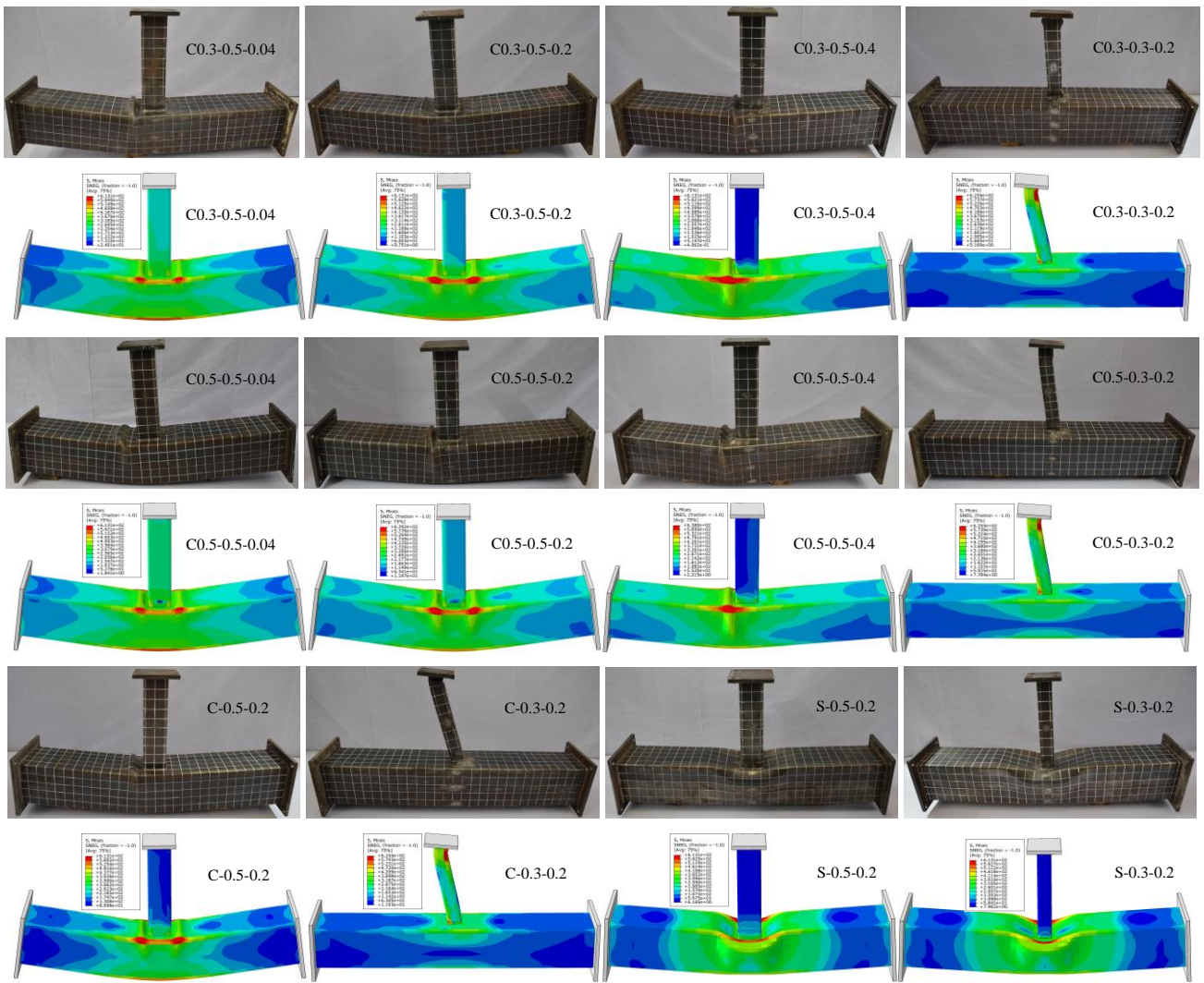
(2) Results without welds

(a) Failure pattern

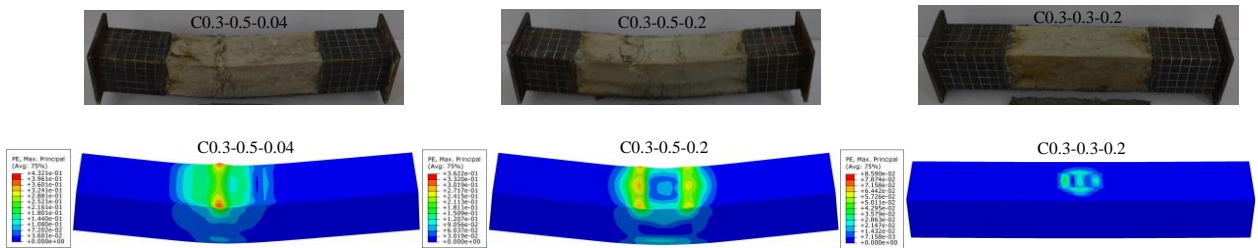


(b)  $F-\Delta_{tb}$  curves

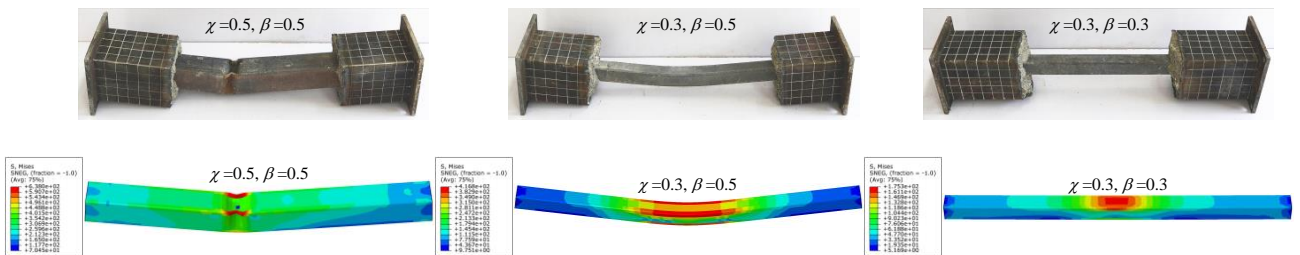
**Fig. 15.** Comparison of FE simulation results of the T-joint specimens with and without welds



(a) Overall pattern

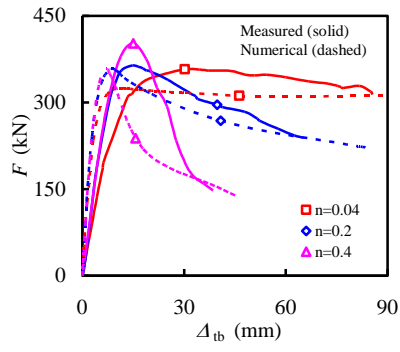


(b) The concrete in the composite chord

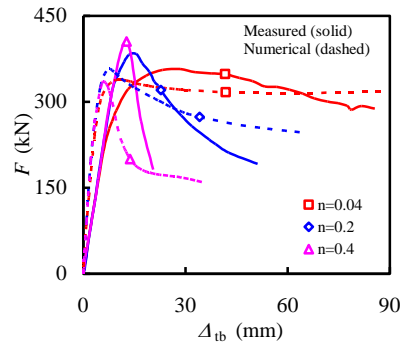


(c) Inner tube of the CFDST chord

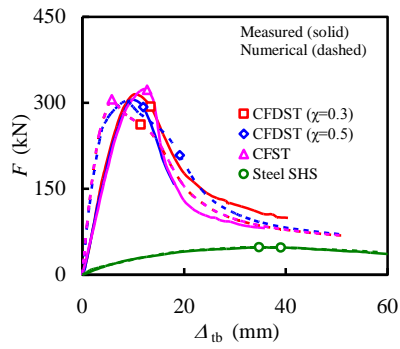
**Fig. 16.** Comparison between the simulated and measured failure patterns



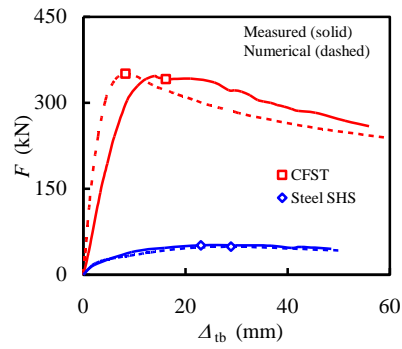
(a) CFDST chord ( $\chi=0.3, \beta=0.5$ )



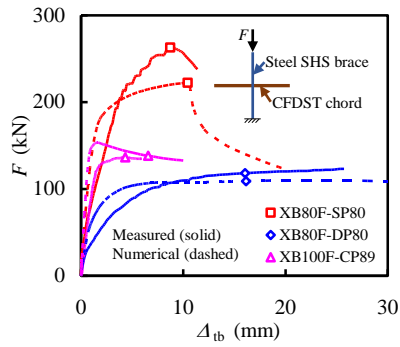
(b) CFDST chord ( $\chi=0.5, \beta=0.5$ )



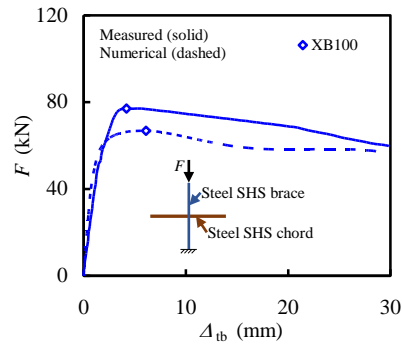
(c) Varied chords ( $\beta=0.3, n=0.2$ )



(d) Varied chords ( $\beta=0.5, n=0.2$ )

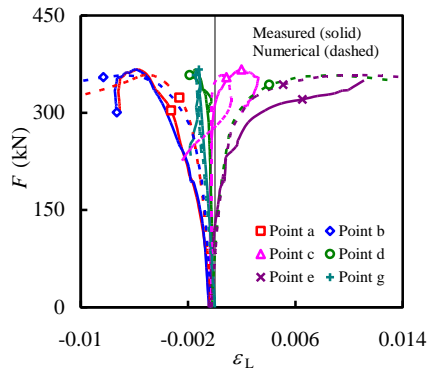


(e) Composite X-joint<sup>[11]</sup>

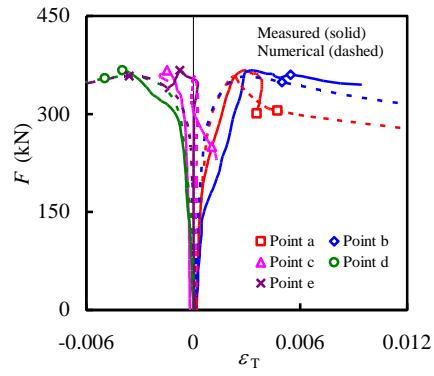


(f) Steel X-joint<sup>[11]</sup>

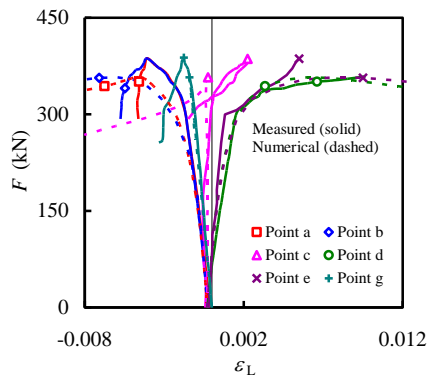
**Fig. 17.** Contrast between the numerical  $F-\Delta_{tb}$  curves and the measured results



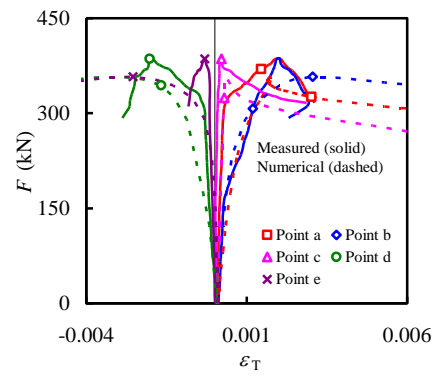
(a)  $\varepsilon_L$  of C0.3-0.5-0.2



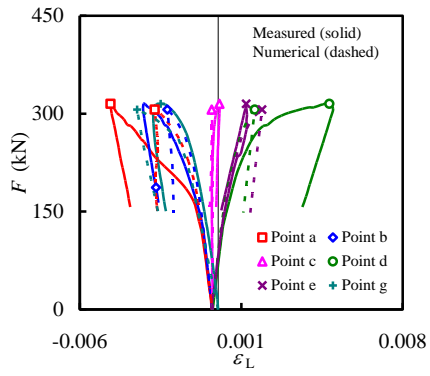
(b)  $\varepsilon_T$  of C0.3-0.5-0.2



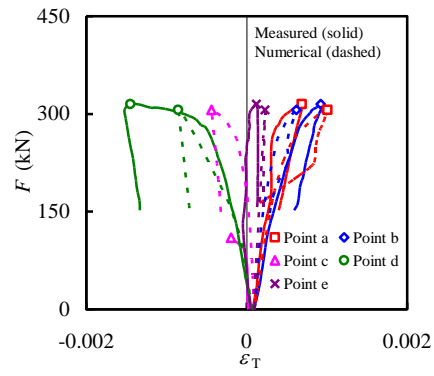
(c)  $\varepsilon_L$  of C0.5-0.5-0.2



(d)  $\varepsilon_T$  of C0.5-0.5-0.2

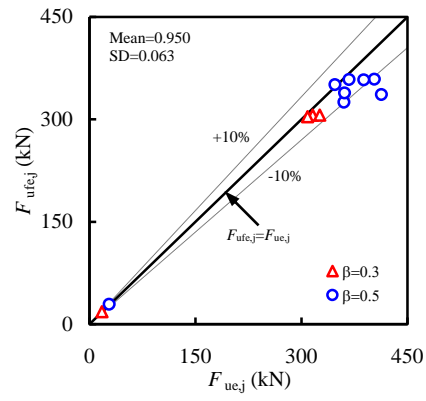


(e)  $\varepsilon_L$  of C0.3-0.3-0.2

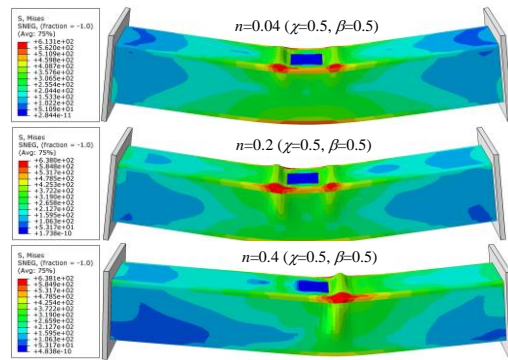
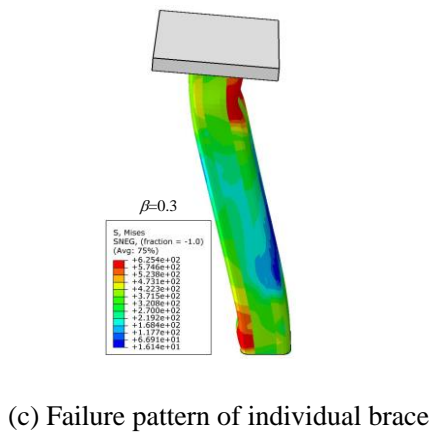
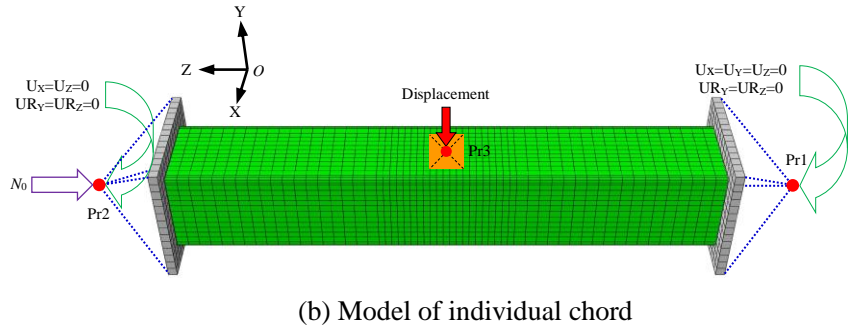
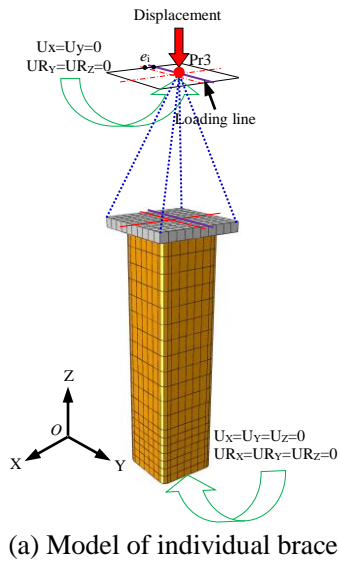


(f)  $\varepsilon_T$  of C0.3-0.3-0.2

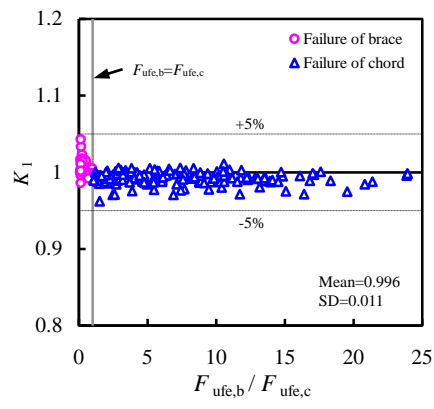
**Fig. 18.** Typical contrast between the numerical and measured  $F$ - $\varepsilon_L$ ( $\varepsilon_T$ ) relationship



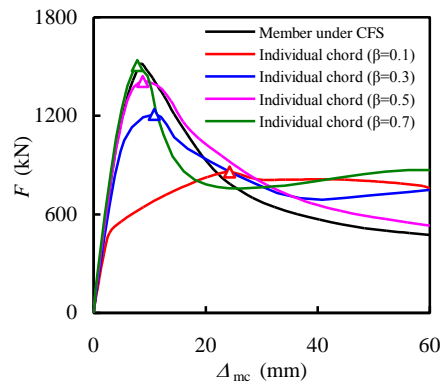
**Fig. 19.** Contrast between  $F_{ufe,j}$  and  $F_{ue,j}$



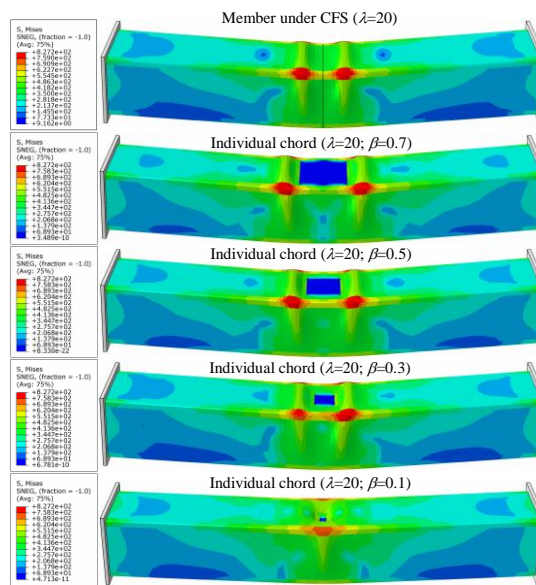
**Fig. 20.** FE modelling of the individual components of the composite T-joints



**Fig. 21.** Variation of  $K_1$  with  $F_{ufe,b}/F_{ufe,c}$  values

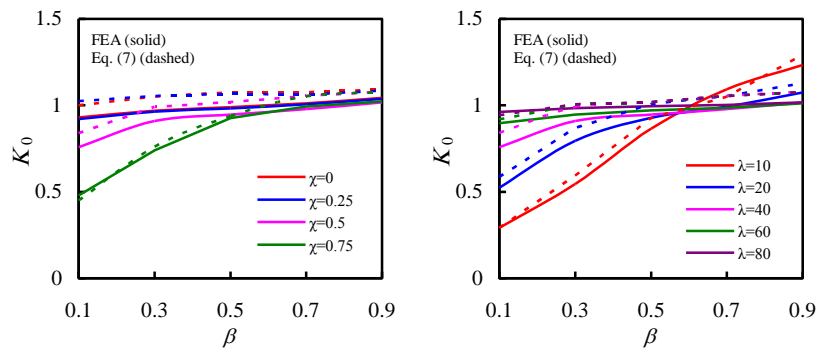


(a)  $F-\Delta_{mc}$  curves



(b) Failure pattern

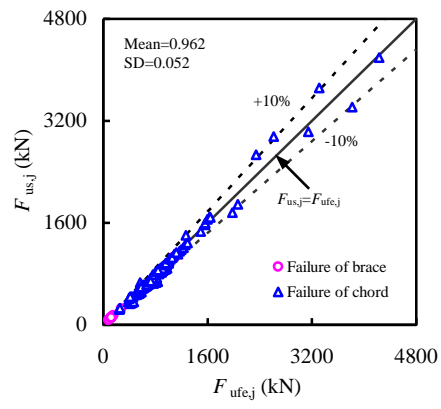
**Fig. 22.** Comparison of square CFST members under different loading conditions



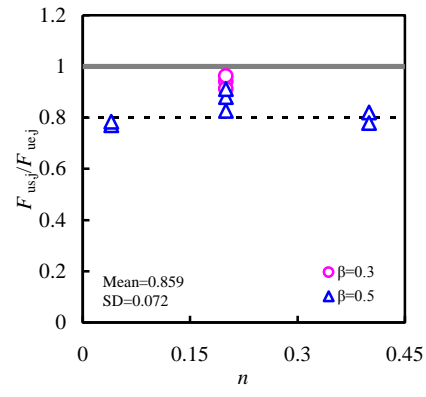
(a) Variation of  $\beta$  and  $\chi$

(b) Variation of  $\beta$  and  $\lambda$

**Fig. 23.** Effect of typical parameters on  $K_0$



**Fig. 24.** Contrast between  $F_{us,j}$  and  $F_{uf,e,j}$



**Fig. 25.** Variation of  $F_{us,j}/F_{ue,j}$  with  $n$  and  $\beta$

**Tables:****Table 1** Properties of steel

Type	Width (mm)	Thickness (mm)	Yield strength (MPa)	Tensile strength (MPa)	Modulus of elasticity (N/mm <sup>2</sup> )	Poisson's ratio	Elongation after fracture (%)
Chord tube	60	2.94	290.9	355.7	1.85×10 <sup>5</sup>	0.318	16.9
	100	3.01	299.3	399.7	1.89×10 <sup>5</sup>	0.292	17.2
	200	4.00	256.0	408.7	1.99×10 <sup>5</sup>	0.343	33.5
Brace tube	60	4.00	298.2	390.1	1.89×10 <sup>5</sup>	0.315	14.8
	100	3.85	294.0	377.6	1.81×10 <sup>5</sup>	0.266	17.1

**Table 2** Information of the specimens

No.	Label	Chord		Brace	$\beta$	$\chi$	$n$	$N_0$ (kN)	$F_{ue,j}$ (kN)	$\Delta_{mc,ue}$ (mm)	$\Delta_{tb,ue}$ (mm)	$F_{ufe,j}$ (kN)	$F_{ufe,j}/F_{ue,j}$	Failure patterns*	
		$b_o \times t_o$ (mm×mm)	$b_i \times t_i$ (mm×mm)	$b_b \times t_b$ (mm×mm)										Observed	Simulated
1	C0.3-0.5-0.04	200×4.00	60×2.94	100×3.85	0.5	0.3	0.04	132.5	360.2	33.3	34.5	325.1	0.902	B+D+F	B+D+F
2	C0.3-0.5-0.2	200×4.00	60×2.94	100×3.85	0.5	0.3	0.2	662.5	367.4	12.4	14.7	358.1	0.975	B+D+F	B+D+F
3	C0.3-0.5-0.4	200×4.00	60×2.94	100×3.85	0.5	0.3	0.4	1325.0	403.1	11.0	14.0	358.9	0.890	B+D+F	B+D+F
4	C0.3-0.3-0.2	200×4.00	60×2.94	60×4.00	0.3	0.3	0.2	662.5	316.0	6.3	10.5	306.3	0.969	A	A
5	C0.5-0.5-0.04	200×4.00	100×3.01	100×3.85	0.5	0.5	0.04	122.9	361.3	27.5	28.4	338.6	0.937	B+D+E	B+D+E
6	C0.5-0.5-0.2	200×4.00	100×3.01	100×3.85	0.5	0.5	0.2	614.5	388.2	12.1	14.5	357.6	0.921	B+D+E	B+D+E
7	C0.5-0.5-0.4	200×4.00	100×3.01	100×3.85	0.5	0.5	0.4	1229.0	413.2	10.9	12.7	336.4	0.814	B+D+E	B+D+E
8	C0.5-0.3-0.2	200×4.00	100×3.01	60×4.00	0.3	0.5	0.2	614.5	308.8	6.0	9.6	304.0	0.984	A	A
9	C-0.5-0.2	200×4.00	--	100×3.85	0.5	/	0.2	661.1	347.6	12.3	13.9	350.8	1.009	B+D	B+D
10	C-0.3-0.2	200×4.00	--	60×4.00	0.3	/	0.2	661.1	325.7	7.4	12.7	306.4	0.941	A	A
11	S-0.5-0.2	200×4.00	--	100×3.85	0.5	/	0.2	169.3	28.1	0.14	3.81	28.9	1.028	C	C
12	S-0.3-0.2	200×4.00	--	60×4.00	0.3	/	0.2	169.3	17.8	0.25	3.38	18.4	1.032	C	C

\*: 'A' stands for local buckling of brace, 'B' stands for local buckling of top flange and webs of outer tube in the composite chord, 'C' stands for yielding of top flange and buckling of webs of steel SHS chord, 'D' stands for compressive crushing and tensile cracking of concrete, 'E' stands for local buckling and overall deflection of inner tube in the composite chord, and 'F' stands for overall deflection of inner tube in the composite chord.

**Table 3** Contrast of bearing capacity of composite T-joint specimens

No.	Label	$F_{ue,j}$ (kN)	$F_{ufe,j}$ (kN)	$F_{ufe,b}$ (kN)	$F_{ufe,c}$ (kN)	$F_{ufe,bc}$ (kN)	$F_{ufe,j}/F_{ue,j}$	$F_{ufe,bc}/F_{ue,j}$
1	C0.3-0.5-0.04	360.2	325.1	485.3	330.2	330.2	0.902	0.917
2	C0.3-0.5-0.2	367.4	358.1	485.3	361.7	361.7	0.975	0.984
3	C0.3-0.5-0.4	403.1	358.9	485.3	363.9	363.9	0.890	0.903
4	C0.3-0.3-0.2	316.0	306.3	307.3	348.0	307.3	0.969	0.972
5	C0.5-0.5-0.04	361.3	338.6	485.3	342.2	342.2	0.937	0.947
6	C0.5-0.5-0.2	388.2	357.6	485.3	362.6	362.6	0.921	0.934
7	C0.5-0.5-0.4	413.2	336.4	485.3	343.8	343.8	0.814	0.832
8	C0.5-0.3-0.2	308.8	304.0	307.3	313.7	307.3	0.984	0.995
9	C0-0.5-0.2	347.6	350.8	485.3	356.2	356.2	1.009	1.025
10	C0-0.3-0.2	325.7	306.4	307.3	343.0	307.3	0.941	0.944
						Mean	0.934	0.945
						SD	0.056	0.054