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Citation: Yang, Y-F., Fu, F. & Liu, M. (2024). Cyclic Behavior of Four-Limbed Circular CFST Latticed Beam- Columns. *Journal of Structural Engineering*, 150(3), 04024006. doi: 10.1061/JSENDH.STENG-13073

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Cyclic Behavior of Four-Limbed Circular CFST Latticed Beam-Columns

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Abstract: Experimental tests to investigate the behavior of four-limbed circular concrete-filled steel tube (CFST) latticed beam-columns under constant axial compression and cyclic lateral force were carried out. Attention was paid to the effect of diameter-to-thickness ratio of limb tube D/T (51.5 and 24.9), axial compression level n (from 0.05 to 0.5) and type of limb (circular CFST and steel circular hollow section (CHS)) on the overall behavior, failure modes and load versus deformation relationship of the specimens. Additionally, the cyclic deterioration of stiffness, ductility and accumulated energy dissipation of the specimens were assessed for seismic design. According to this experimental study, it is found that, due to the interaction between limb tube and its concrete core, the seismic resistance of composite specimens is better than that of steel counterparts. Moreover, the seismic resistance of composite specimens generally reduces with the increase of D/T and n . A finite element (FE) model is further established to replicate the behavior of the specimens, and the simulated cyclic behavior of four-limbed circular CFST latticed beam-columns subjected to constant axial compression and cyclic lateral force agree well with experimental results. Parametric study on the lateral force versus displacement hysteretic curve of four-limbed circular CFST latticed beam-columns was performed using the verified FE model. Finally, an accurate restoring force model (RFM) to predict the lateral force versus displacement relationship of four-limbed circular CFST latticed beam-columns is developed, and the predictions are in good agreement with the numerical and experimental results.

Keywords: Four-limbed circular CFST latticed beam-columns; Cyclic lateral force; Experimental behavior; Finite element simulation; Restoring force model; Metal and composite structures.

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Introduction

Concrete-filled steel tube (CFST) is a composite member formed by filling concrete into thin-walled steel tube, and due to the enhancement of tube wall stability by the filled concrete and the confinement from the outer steel tube on the brittle concrete core, the CFST has the advantages of high strength, good plasticity, superior fatigue/impact resistance, fast-track construction process, and so on (Han et al. 2014; Lin et al. 2021). Recently, the CFST has been widely used in high-rise, long-span, heavy-load bearing, and anti-seismic composite structures (Han et al. 2014; 2020).

In practical engineering, when the slenderness ratio or load eccentricity of a structural member are large, the advantage of good compression performance of single CFST member is difficult to be displayed, and the strength of materials is also not effectively utilized. Under such circumstances, the latticed members consisting of several CFST limbs and the connecting lacings become a better choice to overcome the above-mentioned disadvantages of the single CFST (Matsui and Kawano 1988). The CFST latticed member generally refers to the structural member with the CFST as the limbs and hollow steel tube as the lacings, and the centroid axis of the lacings is in the same plane as that of the two limbs welded with them. According to the difference in the number of the limbs, two-, three- and four-limbed cross-sections are the common form of the CFST latticed members (Yang et al. 2018b). Overall, in comparison with single CFST member having identical load-carrying capacity, the CFST latticed member not only has similar properties, but also possesses lighter weight, greater flexural stiffness and better stability (DBJ/T13-51 2010). The CFST latticed members have been used in practice as beams, columns, piers or arch ribs (Kawano and Sakino 2003; Yang et al. 2018b), and in the future also have good application prospect in large complex engineering both on land and at sea.

During large seismic overloads, the capacity, stiffness, ductility and energy dissipation of the CFST latticed members are dominated by the structural behavior of CFST limbs and tube lacings, as well as the synergistic effect between CFST limbs and tube lacings. Therefore, since the CFST latticed members were first developed, many researchers have paid attention to their seismic behavior by conducting cyclic tests and numerical simulation (Yang et al. 2018b), and the research by Kawano et

al. (1996) is one of earlier attempts to explore the elasto-plastic behavior and deformability of two-limbed circular CFST latticed beam-columns experimentally and theoretically. Table 1 summarizes the available cyclic tests on the CFST latticed beam-columns, in which L is the length of specimens, D and T are the diameter/width and wall thickness of steel circular/square hollow section (CHS/SHS) in the limbs respectively, d and t are the diameter and wall thickness of steel CHS lacings respectively, $f_{y,li}$ is the yield strength of limb tube, f'_c is the cylindrical compressive strength of concrete in the limbs, and n is the axial compression level. It can be seen that, the limbs of most specimens are circular CFST, and the n values are relatively small and have limited variation. Simultaneously, the experimental results show that, the seismic behavior of composite latticed specimens is better than that of steel counterparts; however, further cyclic tests of CFST latticed beam-columns are needed to fully understand the influence of key parameters on their seismic behavior. In addition to experimental studies, finite element (FE) modelling on the cyclic behavior of CFST latticed beam-columns were also carried out by the commercial software ABAQUS (Deng 2012; Huang et al. 2018; Yang et al. 2018b, 2019) and OpenSees (Huang 2015; Yuan et al. 2020). The results showed that, the cyclic behavior of three-limbed circular and four-limbed square CFST latticed beam-columns can be well predicted by the FE model in Yang et al. (2018b, 2019); however, due to the neglect of key influencing factors (e.g. fracture of steel tubes), the cyclic behavior of four-limbed circular CFST latticed beam-columns has not been accurately simulated by the available FE models. Moreover, the restoring force model (RFM) of CFST latticed beam-columns is the basis for nonlinear dynamic time-history analysis of composite structures. Currently, two RFMs for the four-limbed circular CFST latticed beam-columns (Luo 2013; Chen et al. 2014) have been proposed; however, the suitability of them is limited by incomplete tests and/or inadequate FE simulations.

Based on the above review and analysis, it is clear that comprehensive study on the cyclic performance of four-limbed circular CFST latticed members is still limited, which shows that further experiments and theoretical modelling are necessary to guide the engineering practice. In this study, the focus is on the cyclic behaviors of four-limbed circular CFST latticed beam-columns. The effect

of D/T , n and type of limb on the performance of typical specimens under constant axial compression and cyclic lateral force was first investigated, and then a FE model aiming to well predict cyclic behavior of four-limbed circular CFST latticed beam-columns was developed. In addition, a new RFM was proposed on the basis of systematic parametric study, and the accuracy of the RFM was verified by the comparison with the numerical and experimental lateral force versus displacement hysteretic curves.

Experimental program

Information of the Specimens

The behavior of 1/5 scale four-limbed latticed columns in Tianjin International Convention & Exhibition Center or 1/10 scale four-limbed latticed piers in Ganhaizi Bridge (Huang 2015) was investigated experimentally while subjected to combined constant axial compression and cyclic lateral force. A total of eight four-limbed specimens with M-shape layout of the lacings, consisting of six specimens with circular CFST limbs and two specimens with steel CHS limbs as reference, were manufactured, and the lacings of all the specimens were the steel CHS. The design of composite specimens and steel specimens is in accordance to DBJ/T13-51 (2010) and GB50017 (2017), respectively, and takes into account laboratory site conditions, loading capacity of equipment, and limit of research funds. The length (L) of all the specimens was designed to be 1960 mm. The parameters varied in the tests included: 1) D/T , 51.5 and 24.9, which are less than the limit of circular CFST in relevant codes (ACI318-19 2019; ANSI/AISC360-16 2016; EN1994-1-1 2004; GB50936 2014); n , 0.05 (low compressive load), 0.25 (basic compressive load) and 0.50 (high compressive load); and type of limb, circular CFST and steel CHS, in which the values of D/T reflect the impact of material properties and geometric sizes, while the values of n reflect the influence of load level.

The axial compression level (n) of the specimens is expressed as:

$$n = \frac{N_0}{\varphi \cdot N_s} \quad (1)$$

where, φ is the stability factor, N_0 is the constant axial compressive load, and N_s is the sum of the sectional strength of all the limbs, for the specimens with steel CHS limbs, $N_s = \sum_{i=1}^4 f_{y,li,i} \cdot A_{s,i}$, and

for the specimens with circular CFST limbs (Yang et al. 2022), $N_s = k_u \cdot (1.14 + 1.02\xi) \cdot f_{ck} \cdot \sum_{i=1}^4 A_{sc,i}$, in which $A_{s,i}$ and $A_{sc,i}$ are the cross-sectional area of the i th steel CHS limb and circular CFST limb respectively, k_u is the coefficient related to f'_c , when $f'_c \leq 40$ MPa, $k_u=1.1$, and when $f'_c > 40$ MPa, $k_u=1.0$, ξ is the confinement factor of circular CFST (Han et al. 2014), and f_{ck} is the characteristic compressive strength of concrete.

For the steel specimens, the stability factor (φ) related to the equivalent slenderness ratio (λ_e) is determined using GB 50017 (2017), while for the composite specimens, φ is calculated by:

$$\varphi = \frac{1}{2\bar{\lambda}_{sc}^2} \left(k_l - \sqrt{k_l^2 - 4\bar{\lambda}_{sc}^2} \right) \leq 1.0 \quad (2-1)$$

$$k_l = 0.81 + 0.56\bar{\lambda}_{sc} + \bar{\lambda}_{sc}^2 \quad (2-2)$$

$$\bar{\lambda}_{sc} = \frac{\lambda_e}{\pi} \sqrt{\frac{0.67f_{y,li}}{(0.192f_{y,li}/235 + 0.488) \cdot E_{s,li}}} \quad (2-3)$$

where, $\bar{\lambda}_{sc}$ is the nomolized slenderness ratio, and $E_{s,li}$ is the elastic modulus of steel. The equivalent slenderness ratio (λ_e) including the effect of shear deformation of the lacings can be obtained by the formulae in Yang et al. (2019).

The configuration and dimensions of the specimens is shown in Fig. 1, where l_0 is the internode length of the limbs, and h_0 and s_0 are the cross-sectional width along major and minor axis, respectively. The information of the specimens is listed in Table 2, where $P_{u,e}$ is the tested lateral capacity, $K_{a,e}$ is the tested elastic stiffness, and I_d is the ductility index.

The limb tubes and lacings of all the specimens were respectively fabricated by the cold-formed and seamless steel CHS, and each specimen had two square steel plates with the sizes of 520 mm×520 mm×20 mm as the endplates. During the production of the specimens, eight four-limbed steel latticed specimens together with one endplate were first completed according to the designed sizes and welding process. The limbs and the lacings were welded together by the fillet welds, and the welding quality met the relevant codes. There was no eccentricity at the connection nodes of the limbs and the lacings. Fresh concrete was poured into the limbs of six steel specimens to form the composite specimens. In order to guarantee the cohesion of the steel and concrete interface, the end plane of the

concrete infill was polished to be flush with the limb tube before welding another endplate. Moreover, four stiffeners spaced 90 degrees and arranged symmetrically were also welded to the end of each limb tube to avoid unexpected end damage.

The properties of steel CHS in the limbs and steel CHS lacing were respectively obtained by standard tensile and compressive coupons, and the average values of the measured results are listed in Table 3. The mix proportion of concrete in the limbs was: cement=420 kg/m³, fly ash=130 kg/m³, limestone gravel=832 kg/m³, river sand=800 kg/m³, tap water=189.5 kg/m³, and water reducer=6.88 kg/m³. The slump and spread of fresh concrete were 270 mm and 661 mm, respectively. The compressive strength of concrete was obtained by the test on cubes with side length of 150 mm, and the measured average result at 28 days and when conducting cyclic test of composite specimens were 55.2 MPa and 73.8 MPa, respectively. The elastic modulus of concrete was acquired by compressive tests on prisms with side lengths of 150 mm, 150 mm and 300 mm, and the measured average result was 3.47×10^4 N/mm².

Testing Set-up, Loading Histories and Instrumentation

The testing set-up is indicated in Fig. 2. The specimen was treated as a beam-column with one end fixed and the other end free under in-plane cyclic lateral force. A manually controlled hydraulic jack was used to apply the constant axial compressive load (N_0), and a closed-loop servo-controlled hydraulic actuator was employed to apply the cyclic lateral forces/displacements. In order to avoid the overturning effect and ensure the fixed boundary conditions, the bottom endplate of the specimen was connected with a 40 mm thick steel plate by twenty high-strength bolts, whilst the steel plate was fastened to the ground by two rigid beams and four anchor bolts. The top endplate of the specimen was linked to a T-shaped connector with a web thickness of 30 mm by twenty high-strength bolts, and one roller was arranged between the flange of the T-shaped connector and the front end of the actuator to ensure free rotation of the top end of the specimen. The back end of horizontally placed actuator was fixed on the reaction wall, and the front end was connected with the flange of the T-shaped connector by four high-strength bolts. The front end of vertically placed hydraulic jack was

159 acted on a load cell, and another roller through cross-sectional centroid was placed between the load
160 cell and the top of the specimen. The back end of vertically placed hydraulic jack together with a
161 roller system along the loading direction were fixed on a rigid reaction frame, to ensure that the
162 hydraulic jack moved freely with the lateral movement of the specimen.

163 The sequence history of force and displacement, which requires pre-determination of the yield
164 force/displacement of the specimen, was widely used in the cyclic tests of single CFST beam-columns
165 (Varma et al. 2004). However, the applicability of this method to cyclic tests of four-limbed circular
166 CFST latticed beam-columns is unclear, and unsuitable loading regime may lead to premature failure
167 or non-failure of the specimens. Therefore, in order to ensure the stable loading and obtain reliable
168 test results, the displacement-controlled loading protocol was adopted in the present study according
169 to existing guidelines and studies (ATC-24 1992; Yang 2018b, 2019). The history of lateral
170 displacement at the top of the specimen corresponding to the i th loading step (δ_i) is shown in Fig. 3,
171 where N_c is the number of cycles, which are determined with reference to the recommendations in
172 ATC 24 (1992), and δ_0 is the nominal yield displacement, which is set to be identical for easy
173 comparison, i.e. $\delta_0=10$ mm. During the tests, the loading rate was controlled by the frequency of
174 running a complete cycle, that is, when $j \leq 2$, $2 < j \leq 6$, $j=7$, $j=8$, $j=9$ and $j \geq 10$, the loading frequencies
175 were 0.01Hz, 0.008Hz, 0.006Hz, 0.004Hz, 0.003Hz and 0.002Hz, respectively.

176 The generalized displacements were measured by eight displacement transducers (DTs) attached
177 at top endplate and one limb, and the localized strains were measured by five strain gauges pasted on
178 outer wall of two limbs and one lacing, as demonstrated in Fig. 2. In addition, a rigid frame was
179 specially fabricated to hold the displacement transducer in different spatial positions.

180 ***Overall Behavior and Failure Modes***

181 The recording of the tests showed that, for all the specimens, there was no displacement of the bottom
182 endplate, and the connection between the limbs and the endplate was not damaged, indicating that no
183 overturning effect occurred and the fixed boundary conditions were achieved. The failure process of
184 all the specimens was dominated by the damage of the limb tubes, the limb-lacing KT-joints and/or

185 the lacings. Generally, with the increase of δ_i , the local buckling with interval convexity and
186 concavity first occurred near the root of the limbs of two steel specimens, and there were no further
187 damage of lacings when $D/T=51.5$ or a few lacings with coupled global and local buckling when
188 $D/T=24.9$ until the end of the tests. The outward local buckling of limb tube first happened to the
189 root of most composite specimens, except that the cracking of limb tube in the limb-lacing KT-joint
190 first happened to specimen C-B-0.05 and the local/global buckling of lacings happened to specimen
191 C-B-0.25. When the lateral displacement increased to a certain extent, these three kinds of damage
192 coexisted in the composite specimens, and further propagated with the increase of δ_i until the end
193 of the tests. In addition, the good integrity of circular CFST latticed specimens during and after the
194 tests showed that, the coupling behavior of four limbs could be ensured due to the presence of lacings,
195 although the damage occurred to a number of lacings and KT-joints.

196 Fig. 4(a) shows the tested failure modes of the specimens, where the symbols ① through ⑤
197 respectively represent local buckling of limb tube, cracking of limb tube in the KT-joints, coupled
198 global and local buckling of diagonal lacing, cracking/fracture of limb tube and fracture of diagonal
199 lacing. For the steel specimens, the interval local buckling of CHS limbs mainly appears near the root,
200 and further coupled global and local buckling of lacings occurs to the one with a smaller D/T . It
201 should be noted that, the local buckling of limb L_1 in the specimen S-A-0.25 occurs at a location of
202 $(0.09\sim0.14)L$ away from the bottom endplate, which maybe due to the local out-of-arc deformation
203 of the limb tube. However, under the same n value, the composite specimens have more complicated
204 failure modes than steel counterparts, that is, additional cracking of limb tube in the KT-joints and
205 coupled global and local buckling of lacings happen to specimen C-A-0.25, while coupled global and
206 local buckling, even wall fracture, of more lacings occur to specimen C-B-0.25. The cracking of limb
207 tube in the K-joints is mainly caused by the insufficient load-carrying capacity of tube wall under
208 local complex stresses. In general, for the composite specimens, the number of positions with
209 additional failure increases with the improvement of n , which mainly due to the more significant
210 second-order effect under axial compression. Overall, the abovementioned failure modes of four-

limbed circular CFST latticed specimens have also been observed in the previous experiments (Deng 2012; Huang et al. 2018; Chen et al. 2014; Huang 2015; Yuan et al. 2020).

Fig. 5 demonstrates the representative failure modes of concrete core in the CFST limbs. It can be observed that, the failure of concrete core is evidently related to that of limb tube, i.e., the localized compound crushing and cracking are formed at the local buckling position of limb tubes, and the tensile fracture appears at the cracking/fracture position of limb tubes. Simultaneously, the formation of indentation is caused due to the local compression of diagonal lacings near the KT-joints. It can also be seen from Fig. 5 that, there is no obvious wear track at the contact area between the limb tube and its concrete core, that is, no evident slippage occurs to their interface, which is similar to the findings in studies of Han et al. (2009) and Ji et al. (2014), meaning that the limb tube and its concrete core generally function cohesively as a unit.

Variation of Forces and Deformations

Based on the readings of DT₆ and DT₇ in Fig. 2, it was found that, the maximum relative torsion angle of the specimens (i.e. ratio of relative displacement to the distance) varied between 0.23% and 0.70%, indicating that the torsion of the specimens during the tests was limited, and its influence on the variation of forces (deformations) could be ignored.

The recorded lateral displacement along the length of typical specimens based on the readings of DT₀ through DT₅ in Fig. 2 is displayed in Fig. 6, where $\delta_{1,i}$ is the lateral displacement of the i th measuring point in the first cycle of each loading level, h_i is the distance between the i th measuring point and the bottom end, and p is the ratio of lateral force corresponding to $\delta_{1,i}$ to lateral capacity defined later, while a negative p value represents the stage after reaching the lateral capacity. It can be found that, generally, $\delta_{1,i}$ is distributed linearly along the length direction when $p \leq 0.6$ and basically symmetric in two loading directions. When p is between 0.6 and 1.0, the drift angle between two vertical measuring points increases gradually from bottom to top. In the post-peak stage, the distribution characteristic of $\delta_{1,i}$ is similar to that of structural frames with diagonal bracings, i.e. superposition of flexural deformation of four limbs and shear deformation of diagonal lacings.

237 Additionally, while p is the same, the CFST latticed specimen results in a larger $\delta_{1,i}$ than its steel
238 counterpart, namely, a better deformability is achieved.

239 Fig. 7 shows the lateral force (P) versus drift angle (δ_t/L) hysteretic curve of the specimens, where
240 δ_t is the lateral displacement at the top of the specimens. The recorded peak force in two loading
241 directions of the $P - \delta_t/L$ curve is taken as the lateral capacity ($P_{u,e}$). It is indicated that, overall,
242 the $P - \delta_t/L$ hysteretic curve of all the specimens is stable and no pinching characteristics is
243 included. When n is consistent, the load-carrying capacity of steel specimens declines rapidly after
244 reaching $P_{u,e}$ due to the serious local buckling of steel CHS limbs; however, due to the interaction
245 between limb tube and its concrete core, the composite specimens have a wider $P - \delta_t/L$ hysteretic
246 curve and a slower decrease in the load-carrying capacity after achieving $P_{u,e}$ than steel counterparts,
247 exhibiting a better seismic resistance behavior. For the composite specimens, the higher the n value
248 and the smaller the D/T value, the wider the $P - \delta_t/L$ hysteretic curve is. This can be explained
249 that, within a certain range of n values (e.g. ≤ 0.5 in this study), the three-dimensional
250 confinement to concrete core in the limb tube is better than the damage caused by the increase of n ,
251 which leads to a better structural property of circular CFST limbs, and the confinement effect of the
252 tube in the CFST limbs on its concrete core increases with the decrease of D/T (Han et al. 2020).

253 The effect of parameters on $P - \delta_t/L$ backbone curve of the specimens is displayed in Fig. 8. It
254 is shown that, the lateral force (P) in both loading directions generally experiences three stages, i.e.,
255 approximately linear increasing, nonlinear increasing and post-peak declining, as the drift angle (δ_t/L)
256 increases. While n is kept constant, the composite specimens possess a higher elastic stiffness, a
257 larger drift angle corresponding to $P_{u,e}$ and a slower decline rate of load-carrying capacity than the
258 reference steel specimens. In addition, for the composite specimens, with the increase of n and the
259 reduce of D/T , the elastic stiffness and drift angle corresponding to $P_{u,e}$ increase, while the decline
260 rate of load-carrying capacity decreases at the post-peak stage.

261 Fig. 9 exhibits the effect of parameters on lateral force (P) versus strain (ε) hysteretic curves, where
262 ‘L’ and ‘T’ in the brackets represent the longitudinal and transverse strain at the measuring points,

263 respectively, and ε_y is the yielding strain of steel. It can be seen that, irrespective of the type of limb,
 264 the specimens with a smaller D/T generally possess a wider $P - \varepsilon$ hysteretic curve when n is the
 265 same, as the stability of steel CHS limb and the confinement of outer tube in the circular CFST limbs
 266 to its concrete core are enhanced. The strain development at point A is more sufficient than that at
 267 point B because of the difference in bending moments at the two points, while ε_L of the lowest
 268 diagonal lacings gradually develop into complete compression from symmetric coexistence of tension
 269 and compression due to the accumulated damage. In addition, after the tests completed, the maximum
 270 ε_L at points A and C is much larger than ε_y ; however, the maximum ε_L at point B only approaches
 271 ε_y . It is shown in Fig. 9(a) that, the feature of $P - \varepsilon$ hysteretic curve at the same measuring point of
 272 composite specimen is generally similar to that of the reference steel specimen, whilst the former has
 273 a wider $P - \varepsilon$ hysteretic curve than the latter due to the improved performance of circular CFST
 274 limb than steel CHS limb. It is further shown in Fig. 9(b) that, for the composite specimens, the
 275 covered area in the second and fourth quadrants of $P - \varepsilon$ hysteretic curve increases with the increase
 276 of n . The $P - \varepsilon_L$ hysteretic curve is fusiform when n equals to 0.25 and 0.5; however, the $P - \varepsilon_L$
 277 hysteretic curve is pinched in the later period when $n=0.05$, which may be caused by the
 278 cracking/fracture of limb tubes (see Fig. 4).

279 **Mechanical Factors**

280 Fig. 10(a) shows the influence of parameters on lateral capacity of the specimens, where $P_{u,e+}$ and
 281 $P_{u,e-}$ are the lateral capacity in ‘push’ and ‘pull’ directions, respectively, and their average values
 282 ($P_{u,e}$) are given in Table 2. It can be seen that, the lateral capacity of composite specimens is evidently
 283 higher than that of steel counterparts, considering that the load-carrying capacity of the limbs in the
 284 former is higher than that of the limbs in the latter due to the constraint effect of limb tube on its
 285 concrete core. On average, $P_{u,e}$ of composite specimen with D/T of 51.5 and 24.9 is 95.0% and
 286 22.6% higher than that of its steel counterpart, respectively. While n is the same, the composite
 287 specimens with D/T of 24.9 result in a 18.0%-53.7% higher $P_{u,e}$ than those with D/T of 51.5, as
 288 the limb tube having a smaller D/T produces a stronger constraint to its concrete core (Han et al.

2014). For the composite specimens with $D/T=24.9$, $P_{u,e}$ decreases with the increase of n due to the impact of axial compression on the initial cumulative damage and the second-order effect, and $P_{u,e}$ of $n=0.25$ and $n=0.50$ is 5.0-9.6% and 14.6-18.3% lower than that of $n=0.05$, respectively. For the composite specimens with $D/T=51.5$, however, $P_{u,e}$ slightly increases with the increase of n . This may be due to the size deviation caused by the welding of thin-walled steel CHS, which reduces the lateral capacity of the specimens with a relatively small n .

The effect of parameters on elastic stiffness of the specimens measured by the initial slope of the $P - \delta_t$ curve is shown in Fig. 10(b), where $K_{a,e+}$ and $K_{a,e-}$ are the elastic stiffness in ‘push’ and ‘pull’ directions, respectively, and their average values ($K_{a,e}$) are also listed in Table 2. It is shown that, due to the interaction between limb tube and its concrete core, $K_{a,e}$ of composite specimen with D/T of 51.5 and 24.9 equals to 1.207 and 1.234 times that of the reference steel specimen, respectively. When n is the same, the composite specimens with $D/T=24.9$ lead to a 30.2~43.1% higher $K_{a,e}$ than those with $D/T=51.5$, considering that under the same tube diameter a smaller D/T causes a larger cross-sectional moment of inertia and a stronger constraint of limb tube to its concrete core. Furthermore, under the same D/T value, the elastic stiffness of composite specimens reduces when n increases due to the enhanced initial damage under a larger axial compression, and $K_{a,e}$ at $n=0.25$ and $n=0.50$ is 4.2~11.7% and 7.3~15.6% lower than that at $n=0.05$, respectively.

The ductility index (I_d) of the specimens in two loading directions can be computed by the following equation (Yang et al. 2019):

$$I_d = \frac{\delta_{t,0.85}}{\delta_{t,y}} \quad (3)$$

where, $\delta_{t,0.85}$ is the lateral displacement when $P = 0.85P_{u,e+}$ ($P_{u,e-}$) in the post-peak phase of the $P - \delta_t$ backbone curve, and $\delta_{t,y}$ is the yield lateral displacement, which equals to $P_{u,e+}/K_{a,e+}$ or $P_{u,e-}/K_{a,e-}$.

The I_d value of all the specimens is taken as the average value of two loading directions; however, the I_d value of one loading direction is ignored, if the $P - \delta_t$ backbone curve has no descending

stage or lateral force does not drop to $0.85P_{u,e+}(P_{u,e-})$. The obtained I_d of all the specimens is summarized in Table 2. The variation in the I_d value of the specimens is indicated in Fig. 10(c). As can be observed that, due to the combined action between limb tube and its concrete core, the I_d value of composite specimen with D/T of 51.5 and 24.9 is 69.9% and 34.9% higher than that of the reference steel specimen, respectively. In addition, the ductility of composite specimens show a decreasing trend with the increase of n and D/T , because of the existence of the second-order effect under axial compression and the reduction in confinement of concrete core from limb tube, and the I_d value at $n=0.05$ is 1.30~1.37 times and 1.37~1.47 times that at $n=0.25$ and $n=0.50$, respectively, while the I_d value when $D/T=24.9$ is 1.077-1.111 times that when $D/T=51.5$.

323 **Deterioration of Stiffness**

It is well known that, the stiffness of structural beam-columns decreases gradually while subjected to cyclic lateral force. Fig. 11 shows the deterioration of stiffness ($K_i/K_{a,e}$) of the specimens as N_c increases, where K_i is the secant stiffness under the i th loading cycle, and can be determined by the following equation (JGJ/T101 2015):

$$328 \quad K_i = \frac{|+P_i|+|-P_i|}{|+\delta_{t,i}|+|-\delta_{t,i}|} \quad (4)$$

where, $+\delta_{t,i}$ and $-\delta_{t,i}$ are the peak lateral displacement in push and pull loading directions, respectively; and $+P_i$ and $-P_i$ are the lateral force relevant to $+\delta_{t,i}$ and $-\delta_{t,i}$, respectively.

It can be seen from Fig. 11 that, while $N_c \leq 6$, that is, the force reaches about half of lateral capacity, the deterioration of stiffness of composite specimens is slower than that of steel counterparts, mainly because the damage of concrete core is relatively slight during this stage, and the confinement of limb tube to its concrete core makes the performance of circular CFST limbs better than that of steel CHS limbs. While N_c is greater than 6, with the damage aggravation of concrete core under cyclic lateral force, the deterioration of stiffness of composite specimens is quicker than that of the relevant steel specimens. Moreover, when $N_c \leq 6$, the deterioration of stiffness of composite specimens with n of 0.50 is the quickest owing to the most severe initial damage of concrete core, and that of composite specimens with n of 0.05 and 0.25 has no consistent changing characteristics.

340 When N_c is larger than 6, generally, the composite specimens with n of 0.25 and 0.50 have a slower
341 deterioration of stiffness than those with n of 0.05, considering that a higher axial compression
342 results in a greater inhibition of tensile damage of concrete core due to the improvement in the bond
343 between two materials and the compactness of concrete. At the same time, regardless of the type of
344 limb, a quicker deterioration of stiffness is produced for the specimens with a larger D/T due to the
345 reduced performance of the limbs.

346 ***Accumulated Energy Dissipation***

347 The effect of parameters on the relationship between accumulated energy dissipation (E) and drift
348 angle (δ_t/L) is displayed in Fig. 12. It can be observed that, compared with the reference steel
349 specimens, the E value of composite specimens is significantly improved due to their higher lateral
350 capacity, stiffness and ductility. Additionally, under the same conditions, the E value of composite
351 specimens increases with the decrease of D/T and the increase of n . This can be explained that, the
352 smaller the D/T value, the stronger the constraint of limb tube to its concrete core is, meanwhile,
353 the increase of axial compression improves the bond between limb tube and its concrete core and the
354 compactness of concrete core.

355 **Finite Element (FE) Modelling**

356 ***Description of the FE Model***

357 To accurately replicate the cyclic behavior of four-limbed circular CFST latticed beam-columns, a
358 finite element (FE) model was developed using the software package ABAQUS (2014).

359 The limb tubes and lacings in the composite beam-columns were simulated by the metal plasticity
360 model using the von Mises yield criterion and the associated flow rule, and the mixed hardening
361 plasticity model containing isotropic hardening and nonlinear kinematic hardening was utilized to
362 characterize their cyclic behavior. The isotropic hardening was described by the changing of yield
363 surface, and the nonlinear kinematic hardening was expressed by the changing of the back stress
364 vector (ABAQUS 2014; Yang et al. 2018a, 2018b). In this study, the parameters in the mixed
365 hardening plasticity model of steel tube were determined based on the linear interpolation of

366 calibration values in previous tests (Shi et al. 2011; Jia and Kuwamura 2014) and further debugging
 367 by the coincidence between the simulated and measured cyclic behavior of all available circular CFST
 368 latticed beam-column specimens, and the final results are presented in Table 4, where $\sigma|_0$, Q_∞ and
 369 b_s are the parameters in the isotropic hardening model, and $C_{kin,1(\sim 4)}$ and $\gamma_{1(\sim 4)}$ are the
 370 parameters in the nonlinear kinematic hardening model. In addition, the ductile damage model
 371 recommended in ABAQUS (2014) consisting of ductile damage initiation criterion and damage
 372 evolution based on effective plastic displacement was adopted here to simulate the progressive
 373 damage (fracture) of steel tube under cyclic loading, and the details was given in Yang et al. (2019).
 374 Table 5 presents the parameters for ductile damage initiation criterion of steel tube in the specimens,
 375 where K_p is the strengthening coefficient, m_s is the hardening index, and C_1 and C_2 are the
 376 equivalent plastic damage strain under pure shear and uniaxial tension, respectively. Simultaneously,
 377 the measured properties from the tests were used for the elastic parameters of limb tubes and bracings,
 378 whilst the elastic modulus (E_s) and Poisson's ratio (μ_s) were respectively taken as 2.06×10^5 MPa and
 379 0.3 when no test results were available. Moreover, during the tests, the influence of the deformation
 380 of other components (e.g. two endplates, stiffeners and loading plate, etc.) was very limited, and to
 381 simplify the modelling they were treated as the elastic materials with E_s and μ_s of 1.0×10^{12} MPa
 382 and 0.001, respectively.

383 The concrete damaged plasticity (CDP) model in ABAQUS (2014) was selected to model the
 384 complicate cyclic behavior of concrete core in the limb tubes of circular CFST latticed beam-columns.
 385 The recommended formula in ACI 318-19 (2019) was used to obtain the elastic modulus of concrete
 386 (E_c), i.e. $E_c = 4730 \sqrt{f'_c}$, and the Poisson's ratio of concrete was set to be 0.2. The plasticity parameters
 387 of concrete were same as those in Yang et al. (2018b), which were proved to be a good choice for the
 388 FE modelling on cyclic behavior of three-limbed circular CFST latticed beam-columns, that is,
 389 dilation angle=30°, flow potential eccentricity=0.1, ratio of initial equibiaxial compressive yield stress
 390 to initial uniaxial compressive yield stress=1.16, ratio of the second stress invariant on the tensile
 391 meridian to that on the compressive meridian=2/3 and viscosity parameter=0.0005.

392 The relationship between tensile stress and cracking displacement proposed by Goto et al. (2010)
 393 was used as the tensile constitutive model for concrete core in the limb tubes. The FE simulation
 394 results in the literature (Deng 2012; Yang et al. 2018b) demonstrate that, the compressive constitutive
 395 model for the concrete in the steel tube proposed by Han et al. (2007) was capable of well modelling
 396 the properties of concrete core in the limb tubes of circular CFST latticed members under cyclic
 397 loading. In this study, the same model was used for the compressive stress (σ_c)-strain (ε_c) relationship
 398 of concrete core in the limb tubes, and the details are as follows:

$$399 \quad \frac{\sigma_c}{f'_c} = \begin{cases} 2 \varepsilon_c / \varepsilon_p - (\varepsilon_c / \varepsilon_p)^2 & (\varepsilon_c / \varepsilon_p \leq 1.0) \\ \frac{\varepsilon_c / \varepsilon_p}{\beta_0 \cdot (\varepsilon_c / \varepsilon_p - 1)^2 + \varepsilon_c / \varepsilon_p} & (\varepsilon_c / \varepsilon_p > 1.0) \end{cases} \quad (5)$$

400 where, $\varepsilon_p = (1300 + 12.5f'_c + 800\xi^{0.2}) \times 10^{-6}$, $\beta_0 = (2.36 \times 10^{-5})^{[0.25 + (\xi - 0.5)^7]} \cdot (f'_c)^{0.5} / 2 \geq 0.12$.

401 For the CDP model in ABAQUS (2014), the elastic stiffness of concrete is damaged (or weakened)
 402 when the unloading stiffness in the softening stage of stress-strain curve is lower than the initial value,
 403 and in general the damage factors are used to represent the elastic stiffness loss of concrete. In this
 404 study, the suggested formulas by Birtel and Mark (2006) and Goto et al. (2010) were respectively
 405 used to calculate the damage factor (d) of concrete under compression and tension:

$$406 \quad d = \begin{cases} 1 - \frac{\sigma_c \cdot E_c^{-1}}{\varepsilon_c^{pl} \cdot (1/b_c - 1) + \sigma_c \cdot E_c^{-1}} & (\text{under compression}) \\ 1.24(k_t/f'_c) \cdot u_t^{ck} \leq 0.99 & (\text{under tension}) \end{cases} \quad (6)$$

407 where, ε_c^{pl} is the compressive plastic strain; b_c is the ratio of compressive plastic strain to inelastic
 408 strain, which equals to 0.7, k_t is the maximum negative stiffness in the tensile softening stage (Goto
 409 et al. 2010), and u_t^{ck} is the cracking displacement.

410 While subjected to cyclic load, the elastic stiffness of concrete can partially recover. By reference
 411 to the existing results (Yang et al. 2019) and further calibration by the available test results, the
 412 stiffness recovery coefficient under compression (w_c) and tension (w_t) were set to be 0.2 and 0,
 413 respectively, i.e. the stiffness of concrete partially recovers when the stress state changes from tension
 414 to compression, and the stiffness of concrete does not recover when the stress state changes from
 415 compression to tension.

416 The limb tubes and bracings were modelled by the four-node reduced integrating shell elements
417 (S4R), and to meet the calculation accuracy nine Simpson integral points were set along thickness
418 direction of the shell elements. The concrete core in the limbs and other components were modelled
419 using eight-node reduced integral three-dimensional solid elements (C3D8R). The structured
420 meshing was realized by cutting the composite beam-columns into several regions, and the ideal mesh
421 density was determined by gradually refining the meshing until the deviation of the calculation results
422 of two adjacent meshing was less than 5%. In addition, the meshing at the crack prone positions (i.e.
423 root of the limbs and limb-lacing KT joints) of the specimens was refined to capture their fracture
424 process. The meshing of a typical FE model of four-limbed circular CFST latticed beam-columns is
425 demonstrated in Fig. 13.

426 Based on the aforementioned test results, the interface between limb tube and its concrete core was
427 replicated using surface-to-surface contact with finite sliding, which is consistent with the properties
428 of steel-concrete interface under cyclic shear in the tests of Liu et al. (2022). The ‘hard contact’ in
429 normal direction and the ‘Coulomb friction’ in tangential directions, which have been successively
430 used in the FE simulation of composite columns under cyclic loading (e.g. Goto et al. 2010; Ma et al.
431 2018; Yang et al. 2018b), were used in the modelling. After referring to the previous research (Deng
432 2012; Luo 2013; Huang et al. 2018; Yang et al. 2018b) and further verified by the new test results,
433 the tangential friction coefficient between limb tube and its concrete core was taken as 0.6. A
434 consistent mesh density was set at the interface between shell and solid elements to ensure that the
435 elemental nodes of two materials in the same position were coincident. Moreover, on the basis of
436 ensuring accuracy and efficiency, the welding connection between limb tube and bracings/endplates
437 as well as the interface between concrete and endplates were all simplified as the ‘Tie’ constraints.

438 The boundary conditions with one end fixed and the other end free were considered in the FE
439 model of four-limbed circular CFST latticed beam-columns under cyclic lateral force, as shown in
440 Fig. 13. All degrees of freedom of bottom endplate were restricted by setting as ‘ENCASTRE’ to
441 reproduce the fixed boundary conditions achieved in the tests. One reference point (RP1) was set on

the centroid of top endplate, and simultaneously a continuous distribution coupling constraint was established between RP1 and the upper surface of top endplate to restrict the degrees of freedom of RP1 outside the X-Z plane (i.e. $U_Y=U_{R_X}=U_{R_Z}=0$), so as to duplicate the in-plane free boundary conditions. Another reference point (RP2) together with a continuous distributed coupling constraint was set on the side centroid of top endplate to realize the application of cyclic lateral forces/displacements. Two analysis steps were defined, that is, in the first step the constant axial compressive load (N_0) was applied to the RP1, and in the second step the cyclic lateral forces (P) or displacements (δ_t) were applied to the RP2. Furthermore, the geometric nonlinear effects were considered in both analysis steps.

Additionally, the influence of initial geometric defects on the cyclic behavior of four-limbed circular CFST latticed beam-columns was analyzed by using the FE model, and the first buckling eigenmode of limb tubes and lacings in the composite specimens was determined as the initial geometric defects of the corresponding circular CFST latticed beam-columns in the first analysis step, as typically demonstrated in Fig. 14. The comparison between the simulated and measured results shows that, the influence of the initial geometric defects on the FE simulation results is limited when the defect factors are equal to $0.1T$ and $0.01D$ (Lai et al. 2016). As is well known, the welding residual stress has influence on the behavior of steel members. However, the parameters or processes related to the welding were not available for the tested specimens. Therefore, under the premise of ensuring the calculation accuracy, the effect of welding residual stress on the cyclic behavior of four-limbed circular CFST latticed beam-columns was temporarily not considered in the FE simulation.

Validation of FE model

The predicted failure modes of four-limbed circular CFST latticed beam-column specimens and steel counterparts in this study are displayed in Fig. 4(b). The contrast between the predicted and tested results shows that, overall, the failure modes, fracture and deformation of the limbs and out-of-plane deformation of lacings obtained by the FE simulation agree well with the tested results. The FE model is further used to identify the failure modes of four-limbed circular CFST latticed beam-columns. It

is found that, generally, there are two kinds of failure modes, namely compression-shear failure and compression-flexure failure, and the equivalent slenderness ratio (λ_e) is the determining factor. Fig. 15 demonstrates the effect of λ_e on the failure modes of typical four-limbed circular CFST latticed beam-columns. It can be seen that, with the increase of λ_e , the area with high stress in the limb tubes and lacings is concentrated towards the bottom of limb tubes and the peak value of the Mises stress also shows a decreasing tendency, indicating that the compression-shear failure gradually changes to the compression-flexure failure.

The predicted $P - \delta_t/L$ hysteretic curves, lateral displacement distribution, $P - \varepsilon$ hysteretic curves and deterioration of stiffness of four-limbed circular CFST latticed beam-column specimens are compared with the tested results in Figs. 7 and 16, 17, 18, and 19, respectively. In Figs. 19(c and d), to clearly show the comparison results, the curves with $n=0.25$ and $n=0.50$ are shifted to the right by $0.01\delta_t/L$ and $0.02\delta_t/L$, respectively. It is shown that, the predicted results generally accord well with the tested ones. The comparison of mechanical factors of four-limbed circular CFST latticed specimens between the predicted and tested results is shown in Fig. 20, where $P_{u,fe}$ and $K_{a,fe}$ are the predicted lateral capacity and elastic stiffness using the FE model, respectively, and μ and σ denote the mean value and the standard deviation, respectively. The results of statistical analysis exhibit that, in general, the predicted lateral capacity and elastic stiffness of four-limbed circular CFST latticed specimens agree well with the tested results.

The above comparison proves that, overall, the cyclic behavior of four-limbed circular CFST latticed beam-columns can be well predicted by the FE model established in this study.

Restoring Force Model (RFM)

The restoring force model (RFM) is an important basis for the elastic-plastic seismic response analysis of complex members/structures (Fukumoto and Morita 2023). The experimental results of this study show that, the deformation characteristics of a CFST latticed beam-column is superimposed by flexure-type and shear-type under the combined action of constant axial compression and cyclic lateral force, and as a result it is appropriate to use shear layer model to describe its RFM, namely the

494 mathematical model between lateral force and the corresponding displacement.

495 **Key Parameters**

496 The effect of parameters on $P - \delta_t$ hysteretic/backbone curve of four-limbed circular CFST latticed
 497 beam-columns is analyzed using the verified FE model. The basic parameters are: $D \times T = 100 \text{ mm} \times 2.3$
 498 mm , $d \times t = 34 \text{ mm} \times 2.3 \text{ mm}$, $h_0 = s_0 = l_0 = 400 \text{ mm}$, $n = 0.4$, $f'_c = 50 \text{ MPa}$, $f_{y,li} = 355 \text{ MPa}$, yield strength
 499 of lacing ($f_{y,la}$) is 355 MPa , and nominal shear-span ratio $\lambda_m (= L/h_0)$ is 3.0 . The FE modelling
 500 results are demonstrated in Fig. 21, where the black lines denote the relevant backbone curves. It can
 501 be seen that, the trend of $P - \delta_t$ backbone curves and the unloading/reloading criterion of $P - \delta_t$
 502 hysteretic curves are mainly determined by D/T , $f_{y,li}$, f'_c , d/t , n and λ_m . Additionally, $f_{y,la}$
 503 only has an effect on the peak force of $P - \delta_t$ backbone curve of members with a small λ_m .

504 **Detailed RFM**

505 By careful observation and analysis of a large number of FE simulated $P - \delta_t$ hysteretic curves, it
 506 is found that, the RFM of four-limbed circular CFST latticed beam-columns can be denoted by the
 507 segmented lines in Fig. 22, where the line O-A(A')-B(B')-C(C') represents the backbone curve, and
 508 the lines 1-2-1'-2'-1 (before reaching lateral capacity) and 3-4-5-3'-4'-5'-3 (after reaching lateral
 509 capacity) represent the unloading/reloading paths.

510 The Y-coordinate at point B(B') (P_m) on the backbone curve of the RFM, i.e. lateral capacity, is
 511 defined as the force that first satisfies the following conditions: 1) the maximum tensile (shear) strain
 512 of steel tube at the limbs' root reaches 0.01 , and 2) the peak force of $P - \delta_t$ backbone curve. Based
 513 on the regression analysis of the FE simulation results, the formulae of P_m can be expressed as:

$$514 \quad \frac{P_m}{1.17V_u} = \begin{cases} \left[0.35 - 0.43n^{1.9} + \frac{P_m \cdot L}{1.17M_u} \right]^{0.71} & (k_v \leq 0.85) \\ [f(n) \cdot (1.94n \cdot \xi - 1.13n + 1)]^{-0.49} & (k_v > 0.85) \end{cases} \quad (7-1)$$

$$515 \quad V_u = \begin{cases} V_{u,li} + V_{u,la} & (\lambda_m < \lambda_{m0}) \\ M_u/L & (\lambda_m \geq \lambda_{m0}) \end{cases} \quad (7-2)$$

$$516 \quad V_{u,li} = \frac{0.52}{-0.22 + \lambda_m^{0.5}} \cdot (-0.06 + 0.39\alpha_s^{2.57}) \cdot (-3.23 + 0.81\xi^{-0.66}) \cdot N_s \quad (7-3)$$

$$517 \quad V_{u,la} = \frac{0.66 \sum_{i=1}^2 f_{y,la} \cdot A_{s,la,i} \cdot \sin \theta}{L/l_0} \quad (7-4)$$

$$k_v = (V_{u,li} + V_{u,la}) / (M_u / L) \quad (7-5)$$

$$M_u = (0.95 + 0.16\xi^{-0.9}) \cdot r_c \cdot f_{y,li} \cdot \sum_{i=1}^2 1.1A_{s,li,i} \quad (7-6)$$

$$f(n) = 4.54n^2 - 2.8n + 1 \quad (7-7)$$

where, k_v is the shear strength ratio, λ_{m0} is the critical shear-span ratio determining iteratively by $k_v=1.0$, $V_{u,li}$ and $V_{u,la}$ are the shear strength of limbs and lacings, respectively, α_s is the cross-sectional steel ratio of circular CFST limbs (Han et al. 2020), $A_{s,li,i}$ and $A_{s,la,i}$ are the area of the i th limb tube and lacing in the same cross-section, θ is the angle between diagonal lacing and limb, and r_c is the distance between the barycenter of composite section and the centroid of limbs under compression.

The Y-coordinate at point A(A') (P_y) on the backbone curve of the RFM corresponding to the end of elasticity is equal to $0.7P_m$. The initial slope of the backbone curve of the RFM, namely the elastic stiffness (K_a), can be expressed as:

$$K_a = \eta_K \cdot 3 \sum_{i=1}^2 (E_s \cdot I_{s,i} + 0.6E_c \cdot I_{c,i}) / L^3 \quad (8-1)$$

$$\eta_K = \begin{cases} (0.32 - 0.06n) \cdot \alpha_s^{-0.13} \cdot (d/t)^{-0.33} \cdot \lambda_m^{1.13} & (\bar{\lambda}_{sc} \leq 0.46) \\ (1.34 - 0.48n) \cdot \alpha_s^{-0.13} \cdot (d/t)^{-0.13} \cdot \lambda_m^{0.14} & (\bar{\lambda}_{sc} > 0.46) \end{cases} \quad (8-2)$$

where, $I_{s,i}$ and $I_{c,i}$ are the cross-sectional moment of inertia of limb tube and its concrete core in the i th limb about the centroid axis of composite section.

The X-coordinate at point B(B') ($\delta_{t,m}$) on the backbone curve of the RFM can be determined by:

$$\delta_{t,m} = [(6 + 6n^2 - 11.28n + 2.34\xi^{0.54}) \cdot \lambda_m^{-0.69} + 0.06n \cdot \lambda_m] \cdot \frac{P_m}{K_a} \quad (9)$$

The formula for the stiffness at descending stage B(B')-C(C') (K_T) of the backbone curve of the RFM is as follows:

$$K_T = \frac{-0.006\lambda_m^{1.58}\xi^{0.64}}{1+2.54n^2-3.49n+0.66\xi} \cdot K_a \quad (10)$$

The unloading/reloading criterion of the RFM is as follows: 1) when unloading from point 1(3) to point 2(4), the stiffness of linear unloading equals to K_r , and the Y-coordinate at point 2(4) is $\eta_D \cdot P_r$, where P_r is the force at unloading point; 2) linear unloading continues to point 1'(3'), where point 1'(3') is the symmetric point of point 1(3) with respect to the origin of coordinates, and the transition

point 5 is on the backbone curve and its Y-coordinate equals to $\eta_E \cdot P_r$; and 3) reloading continues along path 1'-2'-1 or 3'-4'-5'-3 until the next unloading/reloading cycle begins. The regression formula for the related coefficients is as follows:

$$\frac{K_r}{K_a} = \left(\frac{\delta_{t,m}}{\delta_r} \right)^c \quad (11)$$

$$\eta_D = 0.62n - 0.57[\ln(6.66\alpha_s)]^2 + 0.28 \quad (12)$$

$$\eta_E = [0.017(\ln \alpha_s)^3 + 1.07] \cdot (\lambda_m^{0.12} - 0.14) \cdot [0.18 \ln(n) + 1.17] \quad (13)$$

where, δ_r is the displacement at unloading point, and c is 0.6 and 0 when $\delta_{t,m}/\delta_r < 1.0$ and $\delta_{t,m}/\delta_r \geq 1.0$, respectively.

The comparison of key parameters in the RFM of four-limbed circular CFST latticed beam-columns between the simplified and FE simulation results is shown in Fig. 23, where, the parameters with subscripts 's' and 'fe' are simplified and simulated values, respectively. It can be observed that, generally, the simplified formulae based on regression analysis can well predict the key parameters in the RFM of four-limbed circular CFST latticed beam-columns. The typical comparison between the RFM and the FE/test $P - \delta_t$ hysteretic curves is demonstrated in Fig. 24. It can be seen that, overall, the proposed RFM is capable of well predicting the $P - \delta_t$ hysteretic curves of four-limbed circular CFST latticed beam-columns.

The scope of application of the RFM is: $D/T=22.7-83.3$, $d/t=10-20$, $f_{y,li}(f_{y,la})=235 \text{ MPa}-460 \text{ MPa}$, $f'_c=25 \text{ MPa}-65 \text{ MPa}$, $h_0/l_0=0.67-2.0$, $h_0/s_0=1.0-2.0$, $\lambda_e \leq 80$ and $n \leq 0.6$.

Conclusions

Based on the tests and FE simulation on the cyclic behavior of four-limbed circular CFST latticed beam-columns, the following conclusions can be drawn:

(1) The test results of typical latticed specimens show that, while subjected to constant axial compression and cyclic lateral force, the coupling behavior of four CFST limbs can be ensured due to the linkage of lacings, and the overall failure modes of circular CFST latticed are manifested as cracking of limb tube in the KT-joints due to the insufficient load-carrying capacity of tube wall under local complex stresses, coupled global and local buckling of diagonal lacings and local buckling

569 and/or cracking of limb tubes. Additionally, the localized compound crushing and cracking occur to
570 concrete core in the CFST limbs.

571 (2) Along the length of CFST latticed specimens, the distribution of lateral displacement is
572 analogous to that of structural frames with diagonal bracings. When n is the same, the CFST latticed
573 specimens have a wider $P - \delta_t(\varepsilon)$ hysteretic curve and a slower decline in load-carrying capacity
574 after achieving $P_{u,e}$ than steel counterparts due to the improved performance of circular CFST limbs;
575 while a wider $P - \delta_t(\varepsilon)$ hysteretic curve is produced for the CFST latticed specimens with a larger
576 n and a smaller D/T .

577 (3) While n kept unaltered, the CFST latticed specimen with $D/T=51.5$ results in 95% greater
578 $P_{u,e}$, 20.7% greater $K_{a,e}$ and 69.9% greater I_d than the relevant steel latticed specimen, and for the
579 CFST latticed specimen with $D/T=24.9$, the above enhance percentages are 22.6%, 23.4% and
580 34.9%, respectively. Moreover, $P_{u,e}$, $K_{a,e}$ and I_d of CFST latticed specimens with $D/T=24.9$ are
581 18.0%-53.7%, 30.2%-43.1% and 7.7%-11.1% higher than those of composite specimens with
582 $D/T=51.5$, respectively, and overall the CFST latticed specimens with a smaller n lead to a higher
583 value of $P_{u,e}$, $K_{a,e}$ and I_d .

584 (4) A FE model, which reasonably contains mixed hardening and progressive damage of steel tube
585 as well as damaged plasticity and partial stiffness recovery of concrete core, is established using the
586 ABAQUS, and the FE-simulated cyclic behaviors of two- and four-limbed circular CFST latticed
587 beam-column specimens generally conforms with the experimental observations. The FE simulation
588 results further show that, for four-limbed circular CFST latticed beam-columns, there are two kinds
589 of failure modes, namely compression-shear failure and compression-flexure failure.

590 (5) By the FE modelling, the impact of key parameters on the $P - \delta_t$ hysteretic/backbone curve
591 and unloading/reloading paths is discovered, and the RFM for the four-limbed circular CFST latticed
592 beam-columns is further established based on the regression analysis of the FE simulation results.
593 Generally, the simplified $P - \delta_t$ hysteretic curves according to the RFM are in good agreement with
594 the FE simulation and test results.

595 **Data Availability Statement**

596 Some or all data, models or code that supports the findings of this study are available from the
597 corresponding author upon reasonable request.

598 **Acknowledgements**

599 The studies in this paper are financially supported by the National Natural Science Foundation of
600 China (Projects 51678105). The financial support is gratefully acknowledged.

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717 **Table Caption List**

718 **Table 1.** Summary of available cyclic tests on the CFST latticed beam-columns.

719 **Table 2.** Information of the specimens.

720 **Table 3.** Material properties of steel CHS.

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723

Tables:**Table 1.** Summary of available cyclic tests on the CFST latticed beam-columns.

No.	Section form	Layout of lacings	L (mm)	$D \times T$ (mm \times mm)	$d \times t$ (mm \times mm)	$f_{y,li}$ (MPa)	f'_c (MPa)	n	Number of specimens	Ref.
1	Two-limbed	V-shape	2434	○-60.5 \times 2.3	○-34 \times 2.0	370.4	29.4	0-0.2	8	Kawano et al. (1996)
2	Three-limbed	M-shape	1960	○-100 \times 1.9/4.0	○-34 \times 2.5	303.4/325.2	45.9	0.05~0.52	6	Yang et al. (2018b)
3	Four-limbed	M-shape	1200	○-86 \times 1.5	○-48 \times 2.5	315.0	34.2~50.0	0.2~0.4	8	Deng (2012)
4	Four-limbed	N-shape	1700	○-90 \times 3.4 □-80 \times 3.0	○-42 \times 2.7	259.0	57.9	0.2	1 2	Chen et al. (2014)
5	Four-limbed	-, V-, M- and N-shape	2500	○-110 \times 2.0	○-50 \times 2.0	345.0	32.1~47.5	0.15	6	Huang (2015)
6	Four-limbed	M-shape	1200~3000	○-86 \times 1.5	○-48 \times 2.0~3.0	315.0	16.3	0.2~0.3	4	Huang et al. (2018)
7	Four-limbed	M-shape	1960	□-100 \times 1.8/3.5	□-34 \times 2.5	297.6/275.5	59.3	0.05~0.5	6	Yang et al. (2019)
8	Four-limbed	-shape	2500	○-114 \times 2.0	○-48 \times 2.0	345.0	34.1~54.6	0.15	7	Yuan et al. (2020)

Note: The limb tubes/lacings of each specimen have the same section size and physical properties, and '○' and '□' denote circular hollow section (CHS) and square hollow section (SHS), respectively.

Table 2. Information of the specimens.

No.	Label	$D \times T$ (mm)	$d \times t$ (mm)	λ_e	D/T	n	N_0 (kN)	$P_{u,e}$ (kN)	$K_{a,e}$ (kN/mm)	I_d
1	S-A-0.25	○-100 \times 1.94	○-34 \times 2.46	27.7	51.5	0.25	182.5	66.0	3.38	2.56
2	S-B-0.25	○-100 \times 4.01	○-34 \times 2.46	31.1	24.9	0.25	344.1	140.5	4.72	3.47
3	C-A-0.05	○-100 \times 1.94	○-34 \times 2.46	35.1	51.5	0.05	93.7	117.9	8.45	5.78
4	C-A-0.25	○-100 \times 1.94	○-34 \times 2.46	35.1	51.5	0.25	468.5	128.7	7.46	4.35
5	C-A-0.50	○-100 \times 1.94	○-34 \times 2.46	35.1	51.5	0.50	937.0	131.2	7.13	4.13
6	C-B-0.05	○-100 \times 4.01	○-34 \times 2.46	36.8	24.9	0.05	131.0	181.2	11.00	6.42
7	C-B-0.25	○-100 \times 4.01	○-34 \times 2.46	36.8	24.9	0.25	654.9	172.2	10.54	4.68
8	C-B-0.50	○-100 \times 4.01	○-34 \times 2.46	36.8	24.9	0.50	1309.9	154.8	10.20	4.45

Note: 1) 'S' and 'C' represent steel CHS limb and circular CFST limb, respectively; 2) 'A' and 'B' represent D/T value of steel CHS in the limbs of 51.5 and 24.9, respectively; and 3) the values of 0.05, 0.25 and 0.50 indicate the axial compression level.

Table 3. Material properties of steel CHS.

Type	Diameter (mm)	Thickness (mm)	Yield strength (MPa)	Tensile strength (MPa)	Elastic modulus (N/mm ²)	Poisson's ratio	Elongation (%)
Limb	100	1.94	325.2	413.2	1.80×10^5	0.289	19.9
	100	4.01	303.4	426.2	1.96×10^5	0.274	20.1
Lacing	34	2.46	332.4	452.3	2.00×10^5	0.300	-

Table 4. Parameters in the mixed hardening plasticity model of steel tube.

Type	$D \times T/d \times t$ (mm \times mm)	Isotropic hardening			Nonlinear kinematic hardening								Ref.
		σ_0 (MPa)	Q_∞ (MPa)	b_s	$C_{kin,1}$ (MPa)	η_1	$C_{kin,2}$ (MPa)	η_2	$C_{kin,3}$ (MPa)	η_3	$C_{kin,4}$ (MPa)	η_4	
Limb	60.5 \times 2.3	370.4	21	1.2	8450.2	175.5	7176.9	115.1	2814.3	34.5	1556.2	27.6	Kawano et al. (1996)
Lacing	34 \times 2.0	381.2			8644.6	175.7	7348.6	114.7	2797.4	34.7	1601.4	27.0	
Limb	86 \times 1.5	315.0	21	1.2	7453.0	174.5	6296.0	117.1	2900.9	33.5	1324.5	30.6	Deng (2012)
Lacing	48 \times 2.5												
Limb	110 \times 2.0	345.0	21	1.2	7993.0	175.0	6773.0	116.0	2854.0	34.0	1450.0	29.0	Huang (2015)
Lacing	50 \times 2.0	374.0			8515.0	175.5	7234.1	114.9	2808.7	34.5	1571.3	27.4	
Limb	86 \times 1.5	315.0	21	1.2	7453.0	174.5	6296.0	117.1	2900.9	33.5	1324.5	30.6	Huang et al. (2018)
Lacing	48 \times 2.0/3.0	320.0			7543.0	174.5	6375.5	116.9	2893.1	33.5	1345.5	30.4	
Limb	100 \times 1.94	325.2	21	1.2	7636.6	174.6	6458.2	116.7	2885.0	33.6	1367.2	30.1	This paper
	100 \times 4.01	303.4			7244.2	174.2	6111.6	117.5	2919.0	33.2	1276.0	31.3	
Lacing	34 \times 2.46	332.4			7766.2	174.8	6572.7	116.5	2873.7	33.8	1397.3	29.7	

Table 5. Parameters for ductile damage initiation criterion of steel tube.

Type	$D \times T/d \times t$ (mm \times mm)	K_p (MPa)	m_s	C_1	C_2
Limb	100 \times 1.94	582.76	0.103	0.055	0.222
	100 \times 4.01	652.25	0.145	0.083	0.224
Lacing	34 \times 2.46	680.97	0.135	0.077	0.223

Figures:

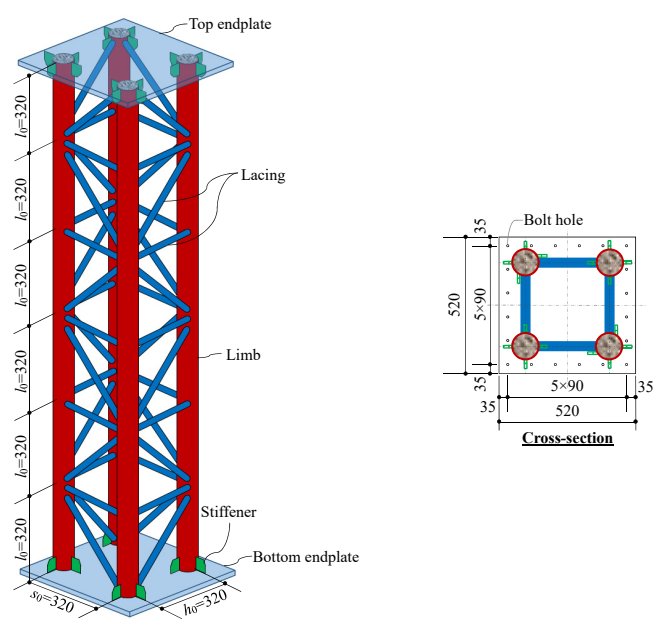


Fig. 1. Configuration and dimensions of the specimens (units are in millimeters).

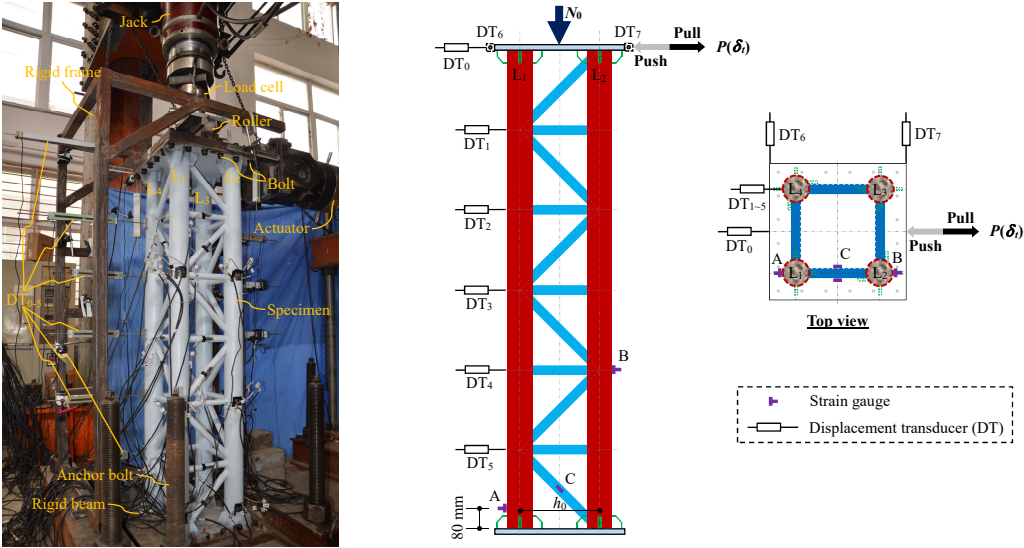


Fig. 2. Testing set-up and instrumentation.

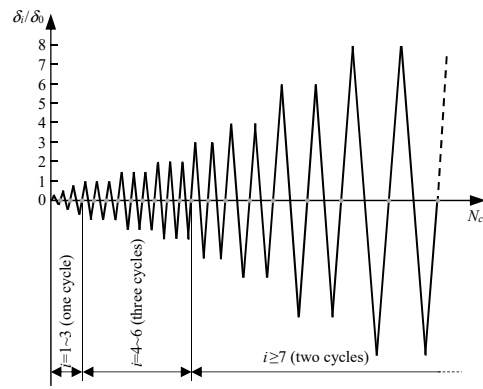
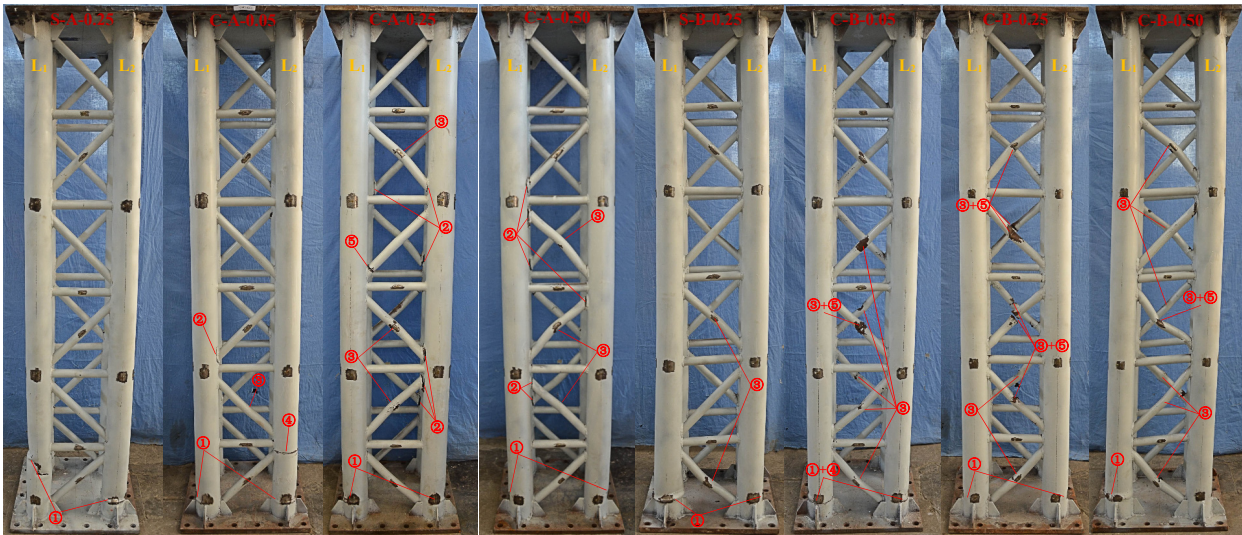
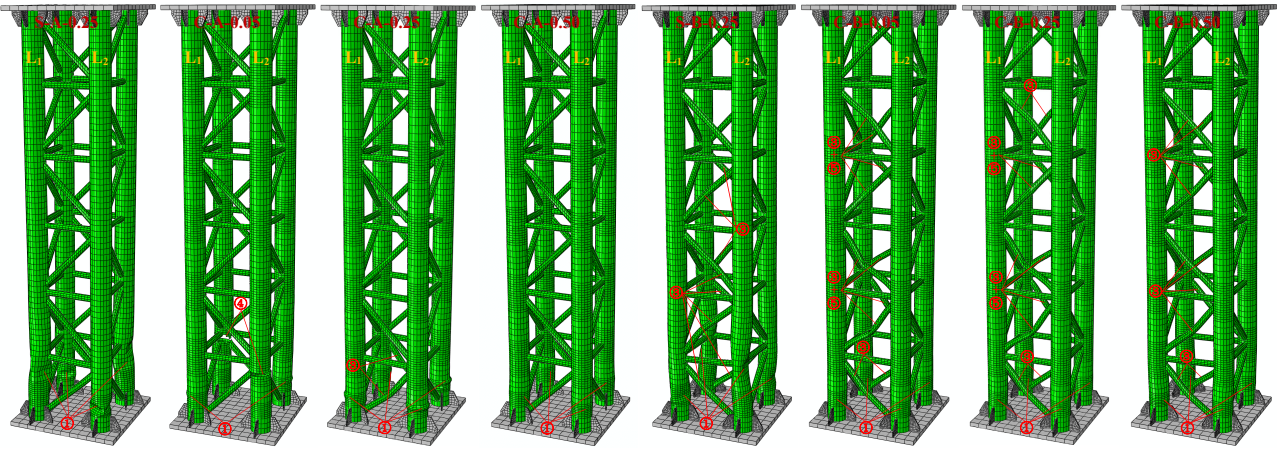


Fig. 3. History of lateral displacement.



(a)



(b)

Fig. 4. Failure modes of the specimens: (a) tested result; and (b) predicted result.

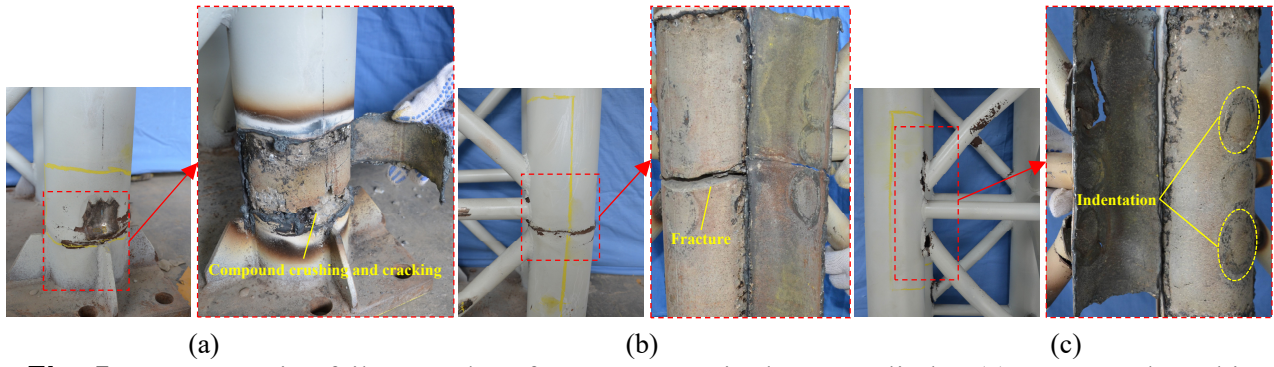


Fig. 5. Representative failure modes of concrete core in the CFST limbs: (a) compound crushing and cracking; (b) fracture; and (c) indentation.

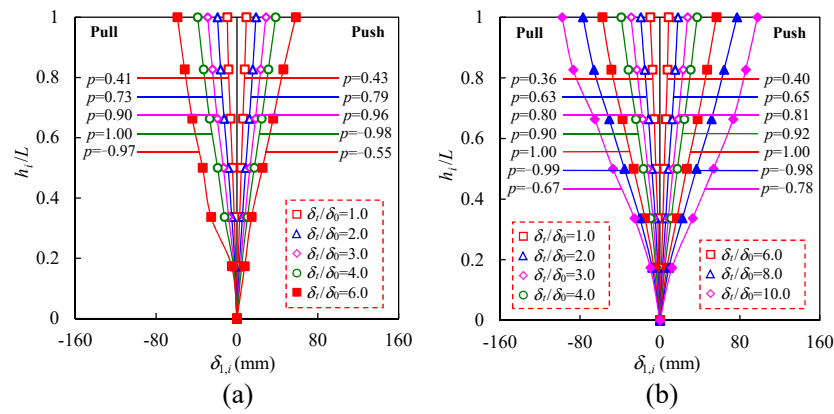


Fig. 6. Lateral displacement along the length of typical specimens: (a) S-A-0.25; and (b) C-A-0.25.

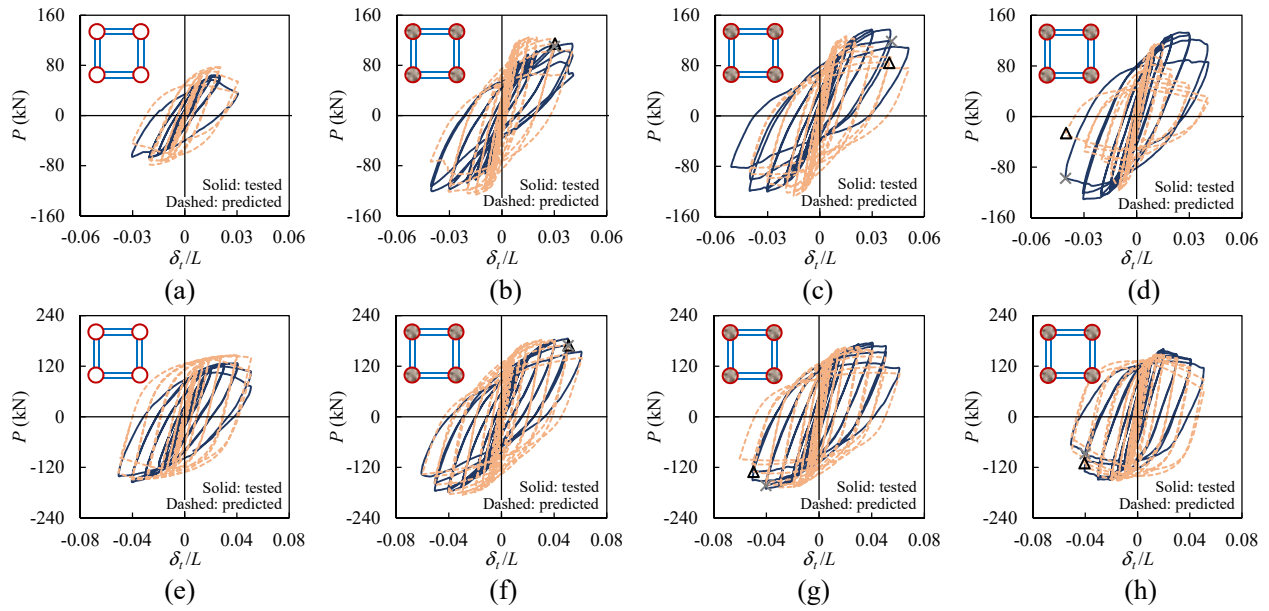


Fig. 7. $P - \delta_t/L$ hysteretic curve of the specimens: (a) S-A-0.25; (b) C-A-0.05; (c) C-A-0.25; (d) C-A-0.50; (e) S-B-0.25; (f) C-B-0.05; (g) C-B-0.25; and (h) C-B-0.50.

(\times : tested starting point of limb tube cracking; and Δ : predicted starting point of limb tube cracking)

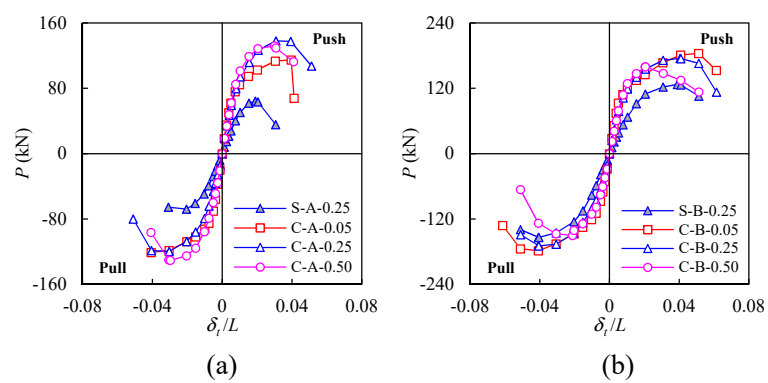


Fig. 8. Effect of parameters on $P - \delta_t/L$ backbone curve of the specimens: (a) $D/T=51.5$; and (b) $D/T=24.9$.

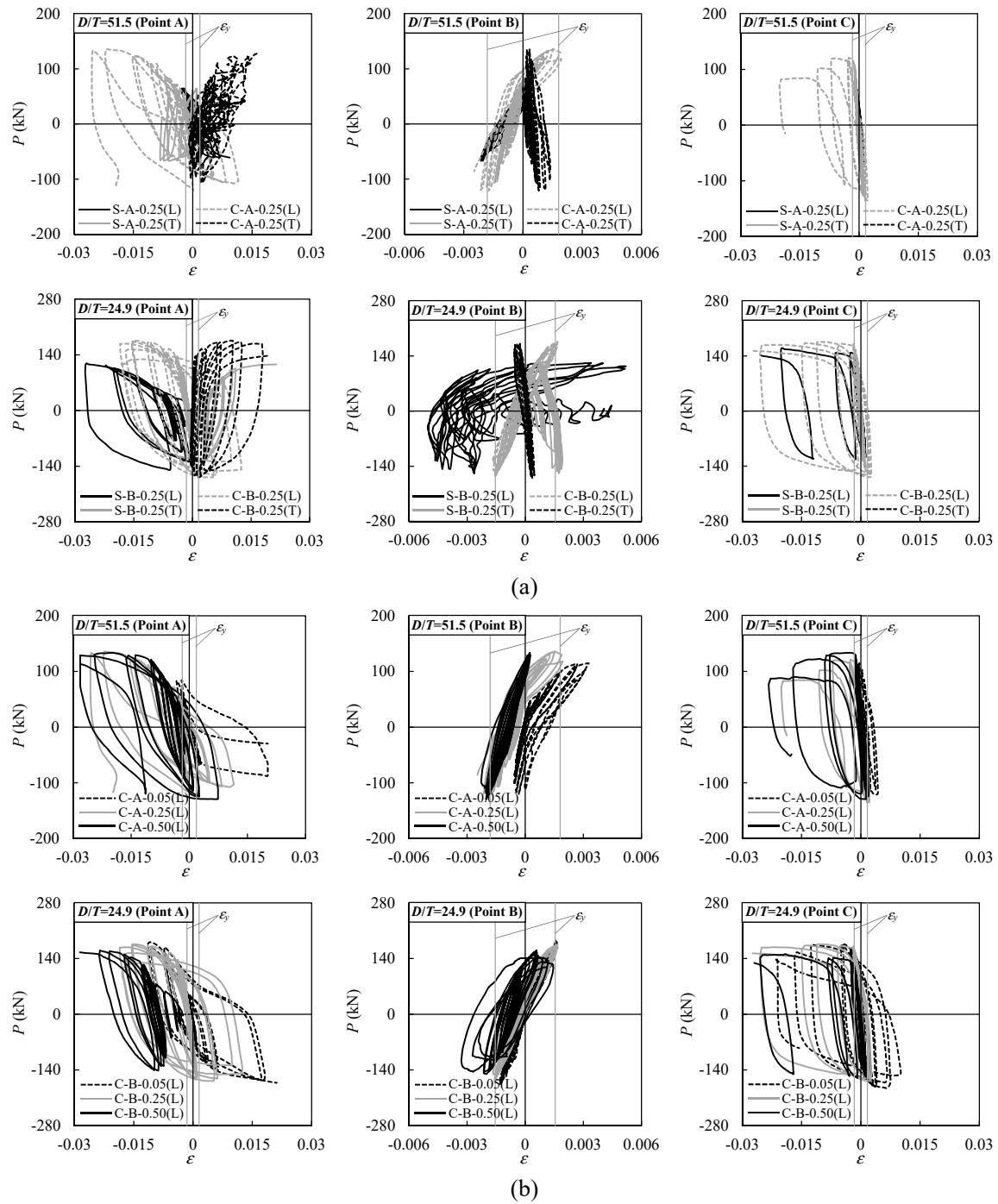


Fig. 9. Effect of parameters on P - ϵ hysteretic curves: (a) type of limb; and (b) axial compression level.

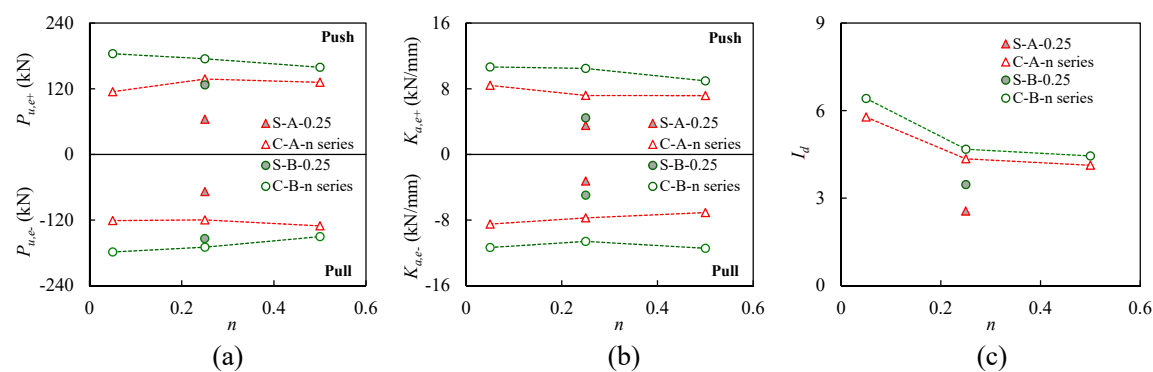


Fig. 10. Variation in mechanical factors of the specimens: (a) lateral capacity; (b) elastic stiffness; and (c) ductility index.

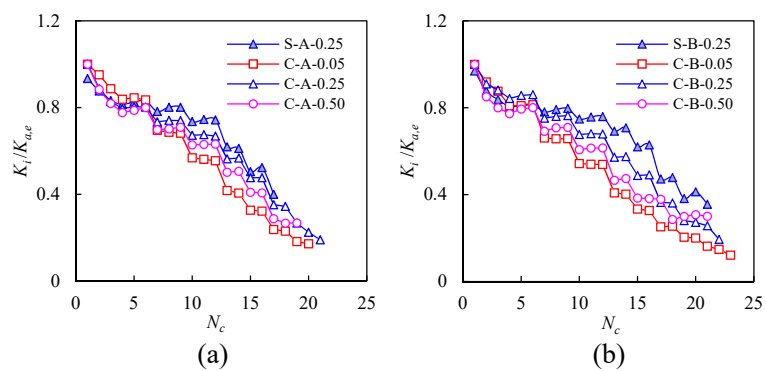


Fig. 11. Deterioration of stiffness of the specimens: (a) $D/T=51.5$; and (b) $D/T=24.9$.

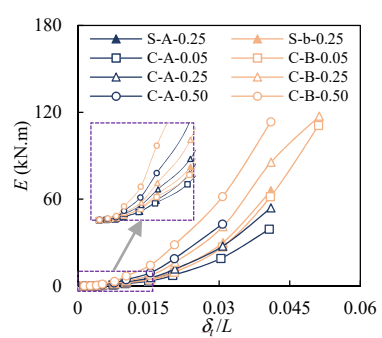


Fig. 12. Effect of parameters on $E - \delta_t/L$ relationship.

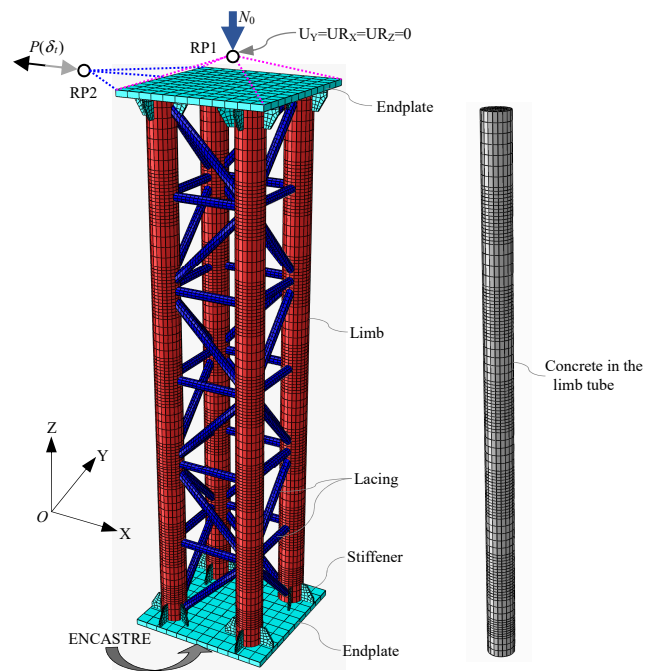


Fig. 13. Meshing and boundary conditions of the FE model.

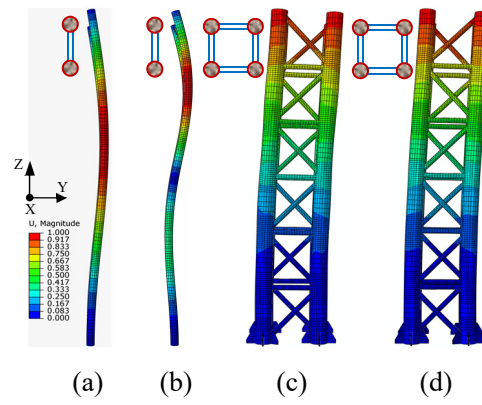


Fig. 14. First buckling eigenmode of limb tubes and lacings in typical composite specimens (Amplification factor=100): (a) CN1R; (b) CB1R; (c) C-A-n; and (d) C-B-n.

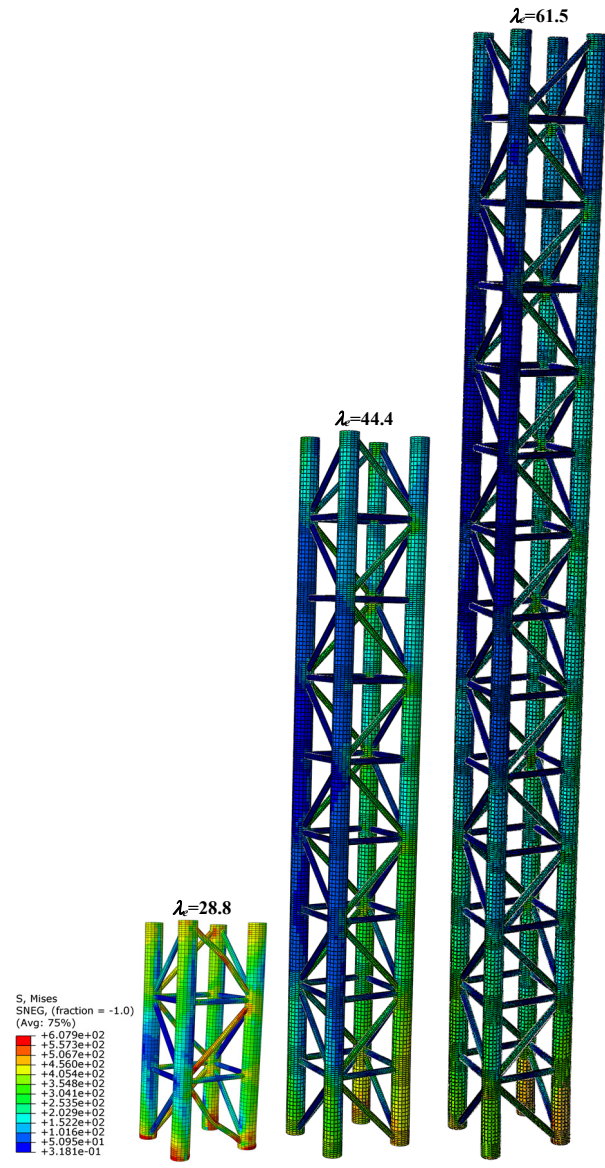


Fig. 15. Effect of λ_e on the failure modes of typical four-limbed circular CFST latticed beam-columns.

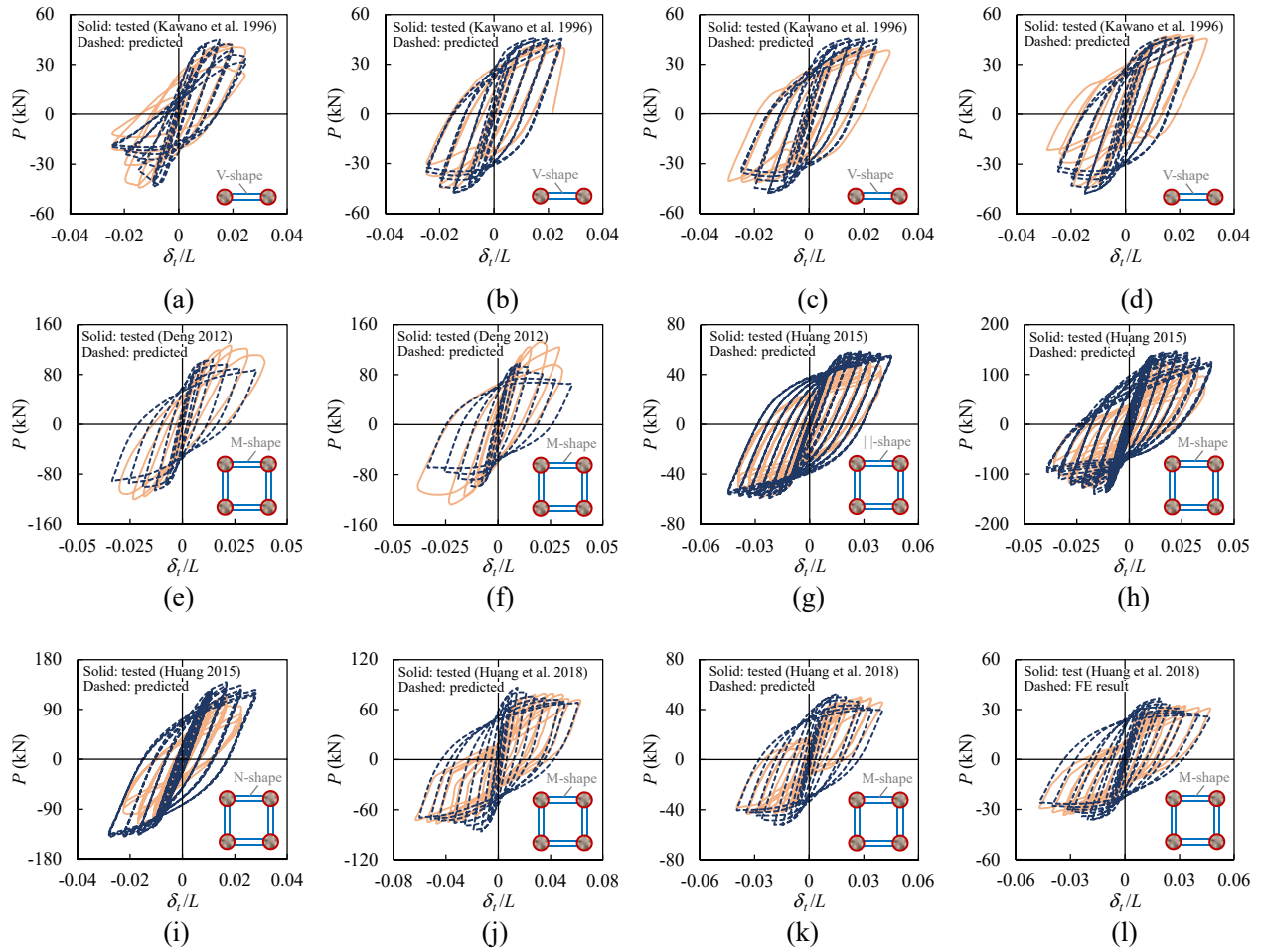


Fig. 16. Typical comparison between the predicted $P - \delta_t/L$ hysteretic curves and the tested results in the literature: (a) CN1R; (b) CB1R; (c) CB0R; (d) CB2R; (e) GZZ1; (f) GZZ3; (g) S1; (h) S5; (i) S6; (j) SCC1; (k) SCC5; and (l) SCC6.

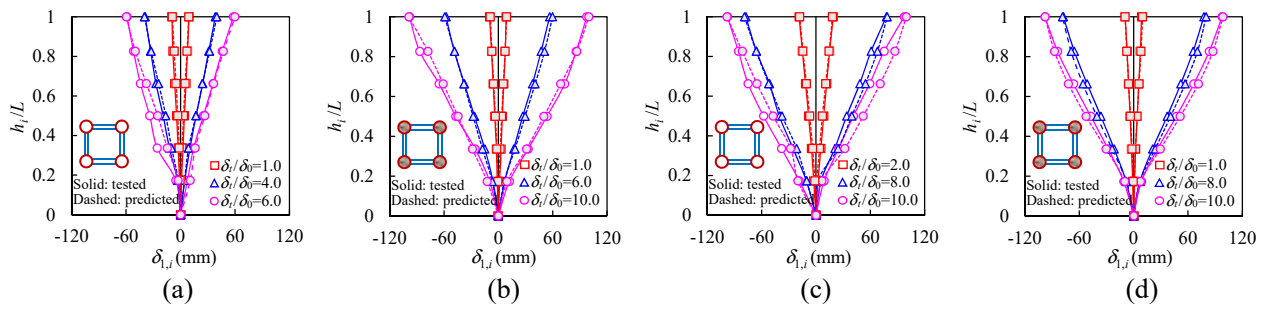


Fig. 17. Typical comparison between the predicted and tested lateral displacement distribution of the specimens: (a) S-A-0.25; (b) C-A-0.25; (c) S-B-0.25; and (d) C-B-0.25.

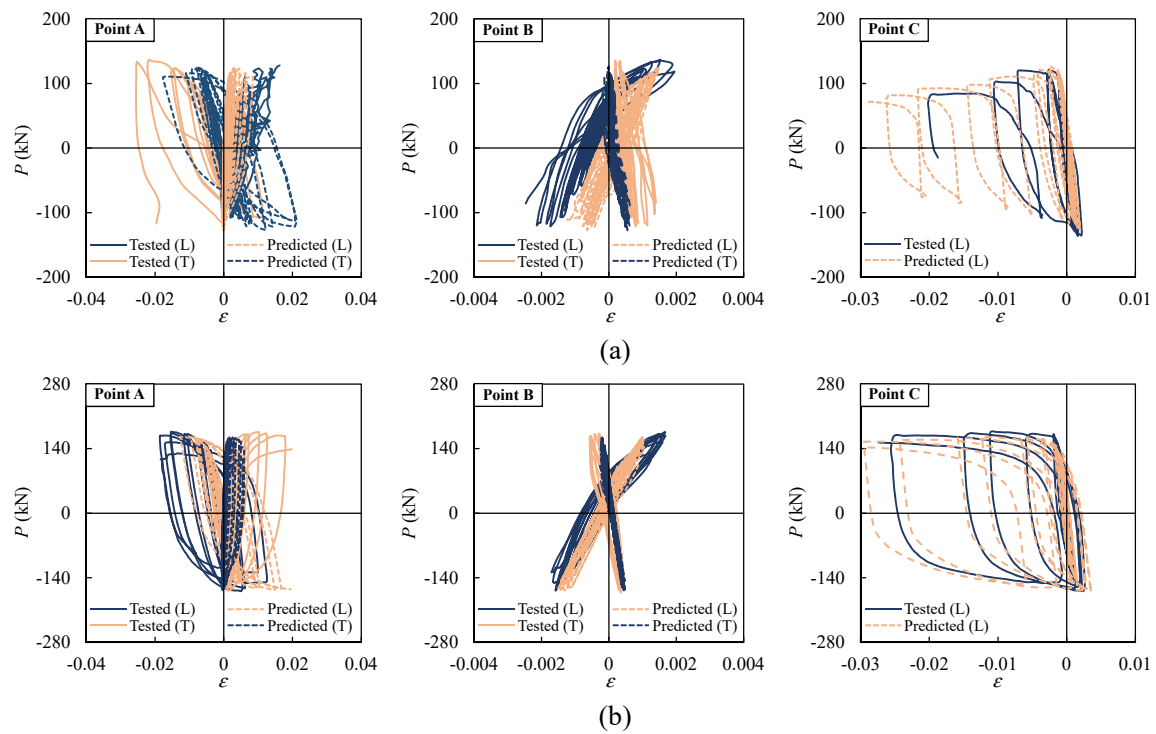


Fig. 18. Comparison between the predicted and tested P - ϵ hysteretic curves of typical specimens: (a) C-A-0.25; and (b) C-B-0.25.

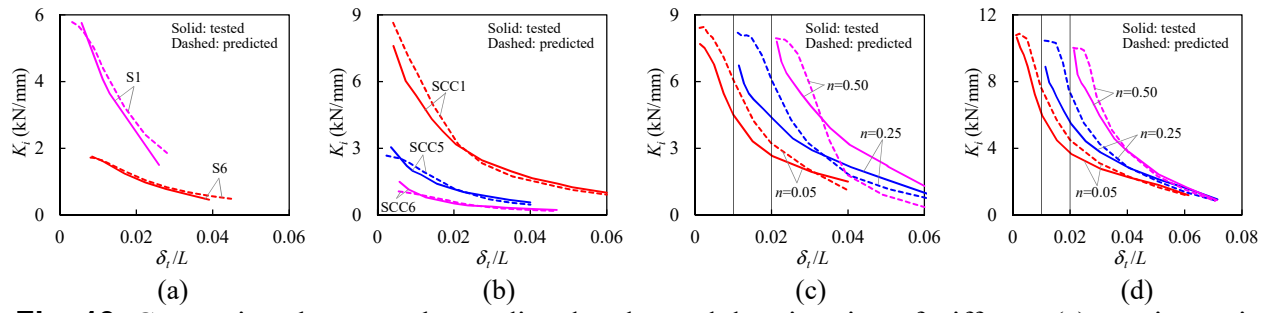


Fig. 19. Comparison between the predicted and tested deterioration of stiffness: (a) specimens in Huang (2015); (b) specimens in Huang et al. (2018); (c) specimens (C-A-n series) in this study; and (d) specimens (C-B-n series) in this study.

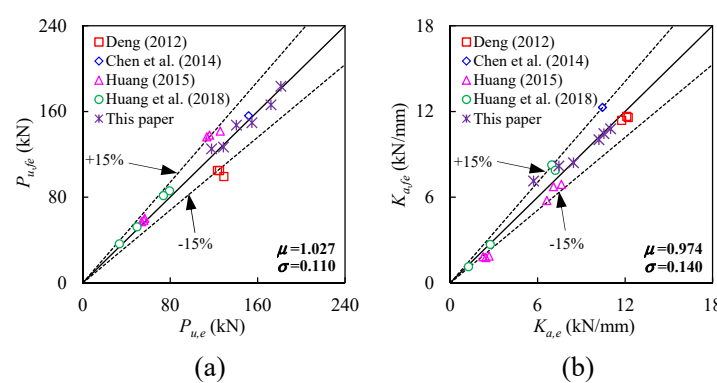


Fig. 20. Comparison between the predicted and tested mechanical factors: (a) lateral capacity; and (b) elastic stiffness.

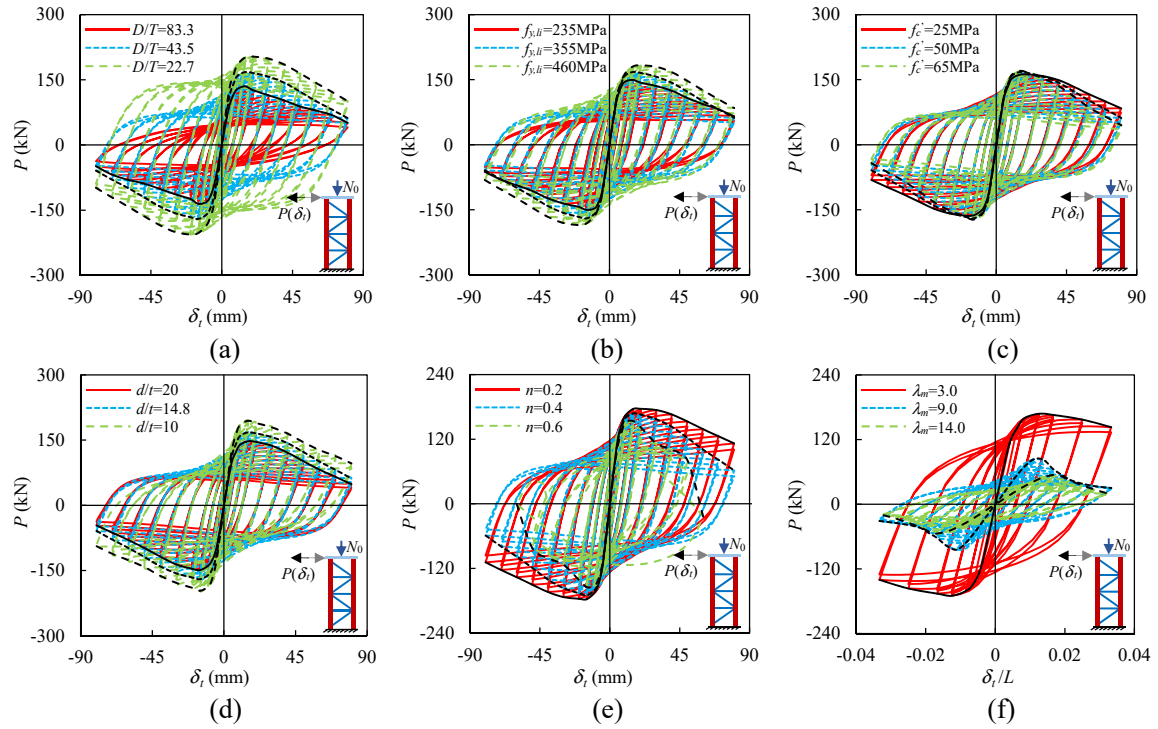


Fig. 21. Effect of main parameters on $P - \delta_t$ hysteretic curve of four-limbed circular CFST latticed beam-columns: (a) D/T ; (b) $f_{y,l}$; (c) f'_c ; (d) d/t ; (e) n ; and (f) λ_m .

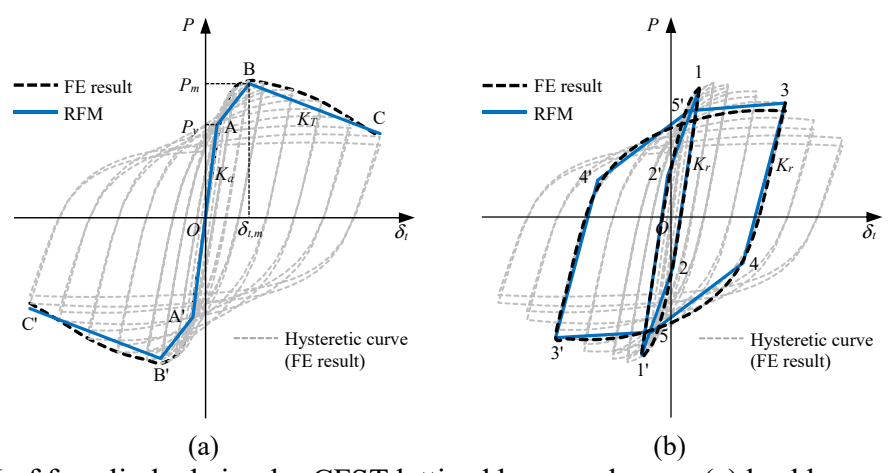


Fig. 22. RFM of four-limbed circular CFST latticed beam-columns: (a) backbone curve; and (b) unloading/reloading paths.

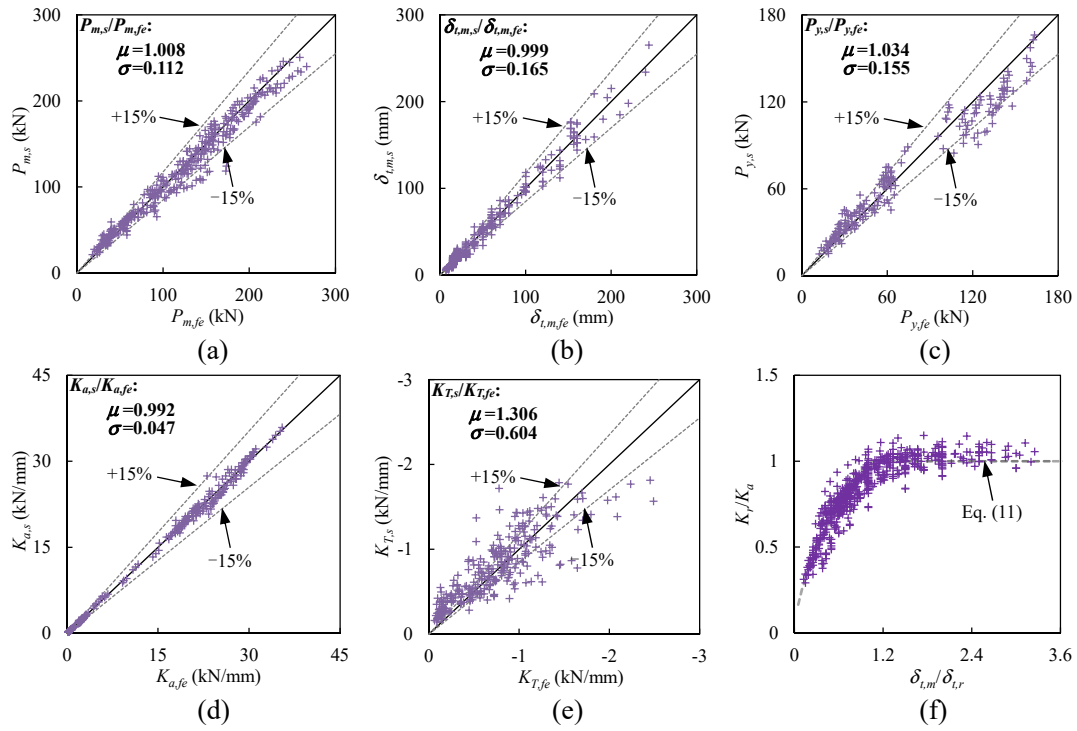


Fig. 23. Comparison of key parameters in the RFM between the simplified and FE simulation results: (a) P_m ; (b) $\delta_{t,m}$; (c) P_y ; (d) K_a ; (e) K_T ; and (f) K_r/K_a - $\delta_{t,m}/\delta_{t,r}$ relationship.

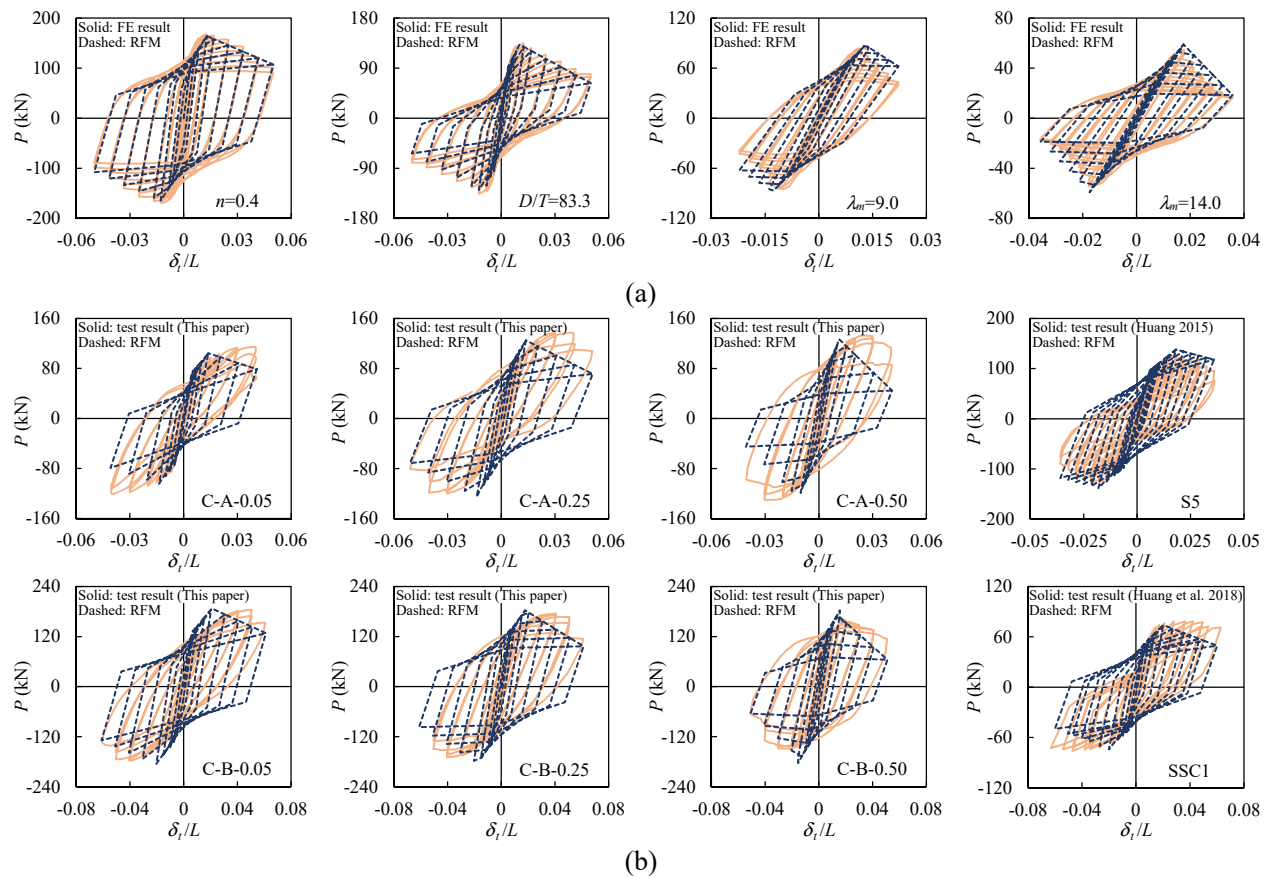


Fig. 24. Typical comparison between the RFM and FE/test $P - \delta_t/L$ hysteretic curves: (a) RFM and FE result; and (b) RFM and test result.

Fig. 1. Configuration and dimensions of the specimens (units are in millimeters).

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Fig. 5. Representative failure modes of concrete core in the CFST limbs: (a) compound crushing and cracking; (b) fracture; and (c) indentation.

Fig. 6. Lateral displacement along the length of typical specimens: (a) S-A-0.25; and (b) C-A-0.25.

Fig. 7. $P - \delta_t/L$ hysteretic curve of the specimens: (a) S-A-0.25; (b) C-A-0.05; (c) C-A-0.25; (d) C-A-0.50; (e) S-B-0.25; (f) C-B-0.05; (g) C-B-0.25; and (h) C-B-0.50.

Fig. 8. Effect of parameters on $P - \delta_t/L$ backbone curve of the specimens: (a) $D/T=51.5$; and (b) $D/T=24.9$.

Fig. 9. Effect of parameters on $P - \varepsilon$ hysteretic curves: (a) type of limb; and (b) axial compression level.

Fig. 10. Variation in mechanical factors of the specimens: (a) lateral capacity; (b) elastic stiffness; and (c) ductility index.

Fig. 11. Deterioration of stiffness of the specimens: (a) $D/T=51.5$; and (b) $D/T=24.9$.

Fig. 12. Effect of parameters on $E - \delta_t/L$ relationship.

Fig. 13. Meshing and boundary conditions of the FE model.

Fig. 14. First buckling eigenmode of limb tubes and lacings in typical composite specimens (Amplification factor=100): (a) CN1R; (b) CB1R; (c) C-A-n; and (d) C-B-n.

Fig. 15. Effect of λ_e on the failure modes of typical four-limbed circular CFST latticed beam-columns.

Fig. 16. Typical comparison between the predicted $P - \delta_t/L$ hysteretic curves and the tested results in the literature: (a) CN1R; (b) CB1R; (c) CB0R; (d) CB2R; (e) GZZ1; (f) GZZ3; (g) S1; (h) S5; (i) S6; (j) SCC1; (k) SCC5; and (l) SCC6.

Fig. 17. Typical comparison between the predicted and tested lateral displacement distribution of the specimens: (a) S-A-0.25; (b) C-A-0.25; (c) S-B-0.25; and (d) C-B-0.25.

Fig. 18. Comparison between the predicted and tested $P - \varepsilon$ hysteretic curves of typical specimens: (a) C-A-0.25; and (b) C-B-0.25.

Fig. 19. Comparison between the predicted and tested deterioration of stiffness: (a) specimens in Huang (2015); (b) specimens in Huang et al. (2018); (c) specimens (C-A-n series) in this study; and (d) specimens (C-B-n series) in this study.

Fig. 20. Comparison between the predicted and tested mechanical factors: (a) lateral capacity; and (b) elastic stiffness.

Fig. 21. Effect of main parameters on $P - \delta_t$ hysteretic curve of four-limbed circular CFST latticed beam-columns: (a) D/T ; (b) $f_{y,li}$; (c) f'_c ; (d) d/t ; (e) n ; and (f) λ_m .

Fig. 22. RFM of four-limbed circular CFST latticed beam-columns: (a) backbone curve; and (b) unloading/reloading paths.

Fig. 23. Comparison of key parameters in the RFM between the simplified and FE simulation results: (a) P_m ; (b) $\delta_{t,m}$; (c) P_y ; (d) K_a ; (e) K_T ; and (f) $K_r/K_a - \delta_{t,m}/\delta_{t,r}$ relationship.

Fig. 24. Typical comparison between the RFM and FE/test $P - \delta_t/L$ hysteretic curves: (a) RFM and FE result; and (b) RFM and test result.