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Experimental behaviour of four-opening concrete filled sandwich steel tube (FCFSST) beam-columns under lateral cyclic loading

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Abstract: In this paper, a new type of composite sections, which are composed of one outer square steel tube, four inner circular or square steel tubes and filled concrete, four-opening concrete filled sandwich steel tube (FCFSST), is first developed. To assess seismic response of typical beam-columns, the tests under axial compression and lateral cyclic loading were conducted on ten FCFSST specimens and one counterpart concrete filled steel tube (CFST) specimen. Effect of several key variables, including void ratio (χ), axial compression level (n), nominal cross-sectional steel ratio (α_n) and sectional type of inner tubes, on the seismic performance of representative FCFSST specimens was investigated in detail. The experimental observations show that, the failure modes of FCFSST beam-column specimens include outward local buckling (LB) and fracture following LB of outer tube, inward LB and fracture following LB of inner tubes and crushing/cracking of sandwich concrete, and generally n and α_n are the controlling factor for the failure of outer tube, whilst χ and sectional type mainly affect the failure of inner tubes. For the FCFSST specimens, a lower n and a larger α_n result in a superior plumpness of the hysteretic curve, a slower stiffness degradation, a higher energy dissipation, and a better ductility, and the influence of n and α_n on key factors is more significant than that of χ . Additionally, in comparison with the reference CFST specimen, the FCFSST specimens show a 20.8% increase in lateral capacity, a 5.0-9.5% increase in cross-sectional flexural stiffness, and a 11.7-21.7% increase in energy dissipation, while presenting a 22.7-25.2% reduction in ductility coefficient and an 8.2-20.1% reduction in ultimate drift, respectively. Finally, the approaches in the existing design specifications of composite structures were used to predict

flexural capacity and cross-sectional flexural stiffness of FCFSSST beam-columns under lateral cyclic loading, and the results indicate that the approach in T/CES can precisely and safely predict the flexural capacity by considering the increased steel yield strength under cyclic loading, whereas the approach in AISC can offer an accurate prediction for the cross-sectional flexural stiffness.

Keywords: Four-opening concrete filled sandwich steel tube (FCFSST); Beam-columns; Lateral cyclic loading; Capacity; Cross-sectional flexural stiffness; Design specification.

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1. Introduction

There have been many applications of concrete filled steel tube (CFST) in practical engineering [1], considering that the structural responses and design methods of CFST structures have been comprehensively recognized in the past decades [2-7]. Nowadays, due to the increased performance requirements, various improved CFST sections have been developed and investigated systematically, including the application of new materials and modified cross-sectional designs, for instance, the CFST members using ultra-high-strength steel tube and concrete infill [8-10] to minimize size and increase elastic deformability, the double-skin CFST (i.e., CDFST) [11-13] and double-opening concrete filled sandwich steel tube (DCFSSST) configurations [14, 15] to reduce self-weight and enhance fire resistance, and fibre-reinforced polymer (FRP) confined concrete [16, 17] or CFST [18, 19] to improve corrosion resistance and/or resistance of thin-walled steel tube to local buckling. The research outcomes [8-15, 18-19] indicate that, compared to the traditional CFST, the improved CFSTs have similar or better structural performance, but their analytical theory and design approach need to be upgraded with the inclusion of some parameters related to the modified characteristics.

A new type of improved CFST, four-opening concrete filled sandwich steel tube (FCFSST), is developed in this study. The FCFSSST member is produced by replacing a portion of concrete core in traditional square CFST member with four symmetrically distributed steel circular hollow sections (CHSs) or square hollow sections (SHSs). It can be judged that, this new type of innovative cross-

section form offers a larger cross-sectional flexural stiffness, a higher bending resistance, a lower self-weight and a better fire resistance than the traditional CFST. While compared to the CFDST, the FCFSST has better flexural performance as its inner tubes are far away from the sectional centroidal axis, and replacing one large-sized inner tube by four small-sized inner tubes enables the modular assembly, thereby achieving cost reduction in transporting and processing of the inner tubes. Simultaneously, the FCFSST has identical performance in both loading directions, which makes it have a wider range of engineering applications than rectangular DCFSST. It is anticipated that, similar to rectangular DCFSST [14, 15], the FCFSST can be utilized as new structural members in high seismic activity regions to withstand extreme loads and contribute high stiffness and stability, such as support columns for offshore platforms, supporting columns on subway platforms, tunnel supports in urban rail transit projects, giant columns for super high-rise buildings, and tower columns for bridges or space structures. The fabrication of new FCFSST mainly includes: connecting the inner tubes first, then connecting the outer tube, and finally casting concrete in sections or as a whole. To facilitate the connection of the segmented steel tubes, the free surface of the inner tubes should be larger than that of the outer tube. Additionally, the FCFSST can be regarded as a new reinforcement technology that employs large-sized steel tubes and filled concrete to retrofit or strengthen existing small-sized tubular members/structures (e.g., lattice columns in factory buildings, pillars in transmission structures, arch ribs in arch bridges), with the aim of effectively enhancing the structural performance and the ability to resist harsh conditions. Before the reinforcement construction, each segment of the new outer tube needs to be divided into two U- or L-shaped parts. The reinforcement sequence is usually to position one part of the outer tube segment and temporarily fix it to the existing tubular members/structures first, then align the other part of the outer tube segment with the fixed part, and finally pour concrete in sections or as a whole.

Currently, there is little research of evaluating structural behaviour of FCFSST members, and the existing studies are limited to the static properties of the improved CFST members with similar composition to the FCFSST. Pi et al. [20] and Guo et al. [21] conducted experimental investigation

into the structural performance of axially compressed CFST short columns with double steel SHSs/CHSs, and further proposed strength calculation formulae for the above three types of composite members. Xu [22] carried out finite element (FE) modelling on axial compressive property of the FCFSSST short columns with outer and inner steel CHSs. Our research group [14, 15] performed experimental study and FE modelling on rectangular DCFSSST short columns under axial compression and beams under bending, and finally suggested the simplified calculation method for their load-carrying capacity.

As it is widely known, when structural members or structures are located in an earthquake zone, it is necessary to conduct reliable seismic performance evaluation and effective seismic design for them. Significant research on the seismic performance of CFDST members has been conducted over the past 20 years. The pseudo-static test, a prevalent method in this field, has been widely adopted to assess the seismic performance of circular-in-circular CFDST beam-columns [23], rectangular-in-rectangular CFDST box piers [24, 25], square-in-square CFDST box columns stiffened with PBL [26], and engineered cementitious composite (ECC) filled high-strength CFDST columns with outer and inner steel CHSs [27]. Furthermore, shaking table tests have been reported on the dynamic behaviour of post-tensioned precast circular-in-circular CFDST piers under tri-directional ground motions [28]. The numerical simulations subsequent to the experimental study have also been carried out to facilitate a deeper comprehension of the seismic performance of CFDST members. For instance, Xia et al. [25] presented a FE model for double-skin steel-concrete composite box (DSCB) piers under lateral cyclic loads, considering the local buckling of steel tubes, and Okamoto and Nakamura [29] conducted seismic analysis of square-in-square CFDST towers in multi-span cable-stayed bridges based on the fiber model method. However, there are no reports on the seismic behaviour of FCFSSST members, which hinders their application in earthquake-prone regions.

The novelty of this work lies in the development of a new type of composite member originated from square CFST, namely FCFSSST. Presented here are the outcomes of an experiment study on the seismic performance and calculation method for mechanical indexes of the proposed composite

beam-columns. From the tests, the typical specimens demonstrate excellent seismic performance. Also, to develop a design method, the axial compression vs. moment correlation curves for CFST and CFDST members are inspected to explore the applicability of them in predicting the flexural capacity of the proposed FCFSSST beam-columns. Moreover, previous studies on the performance of other thematically similar composite members [14, 15, 23-25, 28] offer valuable insights and references for this work, especially regarding the working mechanism and failure modes of different components.

2. Experimental Investigation

2.1. Specimen Design

The objective of this experiment is to evaluate the influence of key variables on the seismic performance of FCFSSST beam-columns and to provide model test data for verifying the accuracy of the design method of mechanical indexes. Eleven composite beam-columns, including 7 FCFSSST specimens containing inner CHSs, 3 FCFSSST specimens containing inner SHSs and one companion CFST specimen, were prepared for the pseudo-static tests under axial compression and lateral cyclic loading. The length of the specimens (L), including the two endplates with a thickness of 20 mm, was 1600 mm. The cross-section of the specimens is depicted in Fig. 1, where B_o is overall width of outer SHS, D_i is overall width (diameter) of inner SHSs (CHSs), t_o (t_i) is wall thickness of outer (inner) tube, and d_e is distance between centroid of two inner tubes.

The key variables in the tests included void ratio (χ), axial compression level (n) and nominal cross-sectional steel ratio (α_n), and the detailed expressions are as follows:

$$\chi = \sqrt{\sum_{i=1}^4 A_{it,i} / A_{ce}} \quad (1)$$

where, $A_{it,i}$ is cross-sectional area within exterior wall of the i th inner tube, and A_{ce} is cross-sectional area within internal wall of outer tube.

$$n = N_0 / N_{cr} \quad (2)$$

where, N_0 is unaltered axial compressive force applied to the specimens, and N_{cr} is stable bearing capacity of the specimens calculated using the formulae for the CFDST and CFST columns in [30].

$$\alpha_n = A_{ot}/A_{ce} \quad (3)$$

where, A_{ot} is cross-sectional area of outer tube.

The specimens in this study were scaled columns used in a subway station platform with a scale ratio of about 1:5, and were designed in accordance to relevant specifications, namely T/CCES 7-2020[30] and AISC 341-16[31], and previous research [23]. It is well known that the seismic performance of structural members is significantly affected by load, material and geometric parameters. In this study, the selected variable n reflects the change in load, while the selected variable α_n exhibits the change in material and geometric dimensions. The ranges of n (0.05-0.50) and α_n (0.068-0.111) generally correspond to the typical column configuration and seismic design strategy in engineering practice [23, 30, 31]. The selection of specific materials was based on T/CCES 7-2020[30] and took into account the capacity of testing devices. Furthermore, considering the cross-section form of the FCFSSST, the void ratio (χ) is a potentially significant factor affecting its seismic performance. According to the limit for the void ratio of CFDST in [30], a sufficient gap between inner and outer steel tubes is required to ensure the pouring quality and mechanical performance of concrete, and the maximum void ratio of the specimens was finally determined and set as 0.50, with a variation range of 0.22 to 0.50.

The details of the specimens are summarized in Table 1, where $f_{yo}(f_{yi})$ is yield strength of outer (inner) steel tube, Q_{LB} is initiation load of local buckling of outer tube, Q_{ce} is lateral capacity, $(EI)_e$ is cross-sectional flexural stiffness, μ is ductility coefficient, θ_u is ultimate drift, and $E_{a,s}$ is accumulated energy dissipation when the load drops to $0.85Q_{ce}$. In addition, each specimen is assigned an ID, starting with the type of cross-section, where ‘C’ stands for the FCFSSST specimen having inner CHSs, ‘S’ stands for the FCFSSST specimen having inner SHSs and ‘Ref’ stands for the reference CFST specimen, together with overall width or diameter of inner tubes, followed by the nominal wall thickness of outer tube, and ending with the axial compression level.

2.2. Material Properties and Specimen Production

Seven different types of steel tubes were used to make the outer and inner tubes of the specimens.

Three standard coupons were tested under axial tension to obtain the properties of the steel tube, which were derived from the average of the measured results, as presented in Fig. 2 and Table 2. A commercially ready-mixed concrete with a design strength grade of C40 was adopted to fill the gaps between the inner and outer steel tubes. The concrete mixture was prepared with the following materials as specified by the supplier: Ordinary Portland Cement (P.O 42.5R), crushed granite coarse aggregate with a maximum size of 20 mm, river sand, fly ash (Grade I), tap water, and polycarboxylic acid series of high-performance water reducer. The compressive strength of concrete after curing for 28 days ($f_{cu,0}$) and during cyclic tests (f_{cu}) of composite specimens were measured by a sufficient number of cubes having sides of 150 mm. The axial compressive tests of three prismatic blocks were conducted to obtain elastic modulus of concrete (E_c). The slump and spread were also recorded before casting the concrete. The mixture proportion and properties of concrete is presented in Table 3, where f_{co} is the characteristic compressive strength converted from f_{cu} according to Eurocode.

The 'direct square forming' process was employed to fabricate the outer tubes. In this process, a hot rolled steel plate with length of 1600 mm was directly bent into a square tube at room temperature, and the final cold-formed steel SHSs consisted of a direct butt weld located in the middle of the side width. Meanwhile, the finished cold-formed steel SHSs or CHSs were used as the inner tubes. There were two endplates for each specimen, one of which was machined with four holes adapted to the outer diameter of the inner tubes, and the edge of the holes was beveled to facilitate reliable welding between the endplate and the inner tubes. Before placing the sandwich concrete, the endplate without holes was sequentially welded to the inner tubes and the outer tube using the fillet welds. The curing of the sandwich concrete was accomplished in an outdoor open space. To maintain the moist state during the curing period, the exposed portion of the sandwich concrete on the top of the specimens was covered with cotton cloth and watered regularly. After curing for 14 days, the sandwich concrete was smoothed against the upper surface of outer tube, and the endplate with four holes was welded to both inner and outer tubes. Finally, to prevent the occurrence of end destruction during the loading process, eight stiffeners were welded to each endplate and the outer steel SHS simultaneously.

2.3. Testing Set-up and Instrumentation

The composite specimens in this study were designed to replicate the frame columns between the inflection points of adjacent floors, and the inter-story shear induced by an earthquake was transferred from the rigid floor to the column at the floor level [32]. As depicted in Fig. 3, the composite specimen was horizontally arranged with both endplates connected to a hinge through high-strength bolts. One hinge together with two pin plates were fixed by connecting with a reaction block, and the other hinge together with two pin plates were connected to a set of devices for applying axial forces. The constant axial compression was imposed using a hydraulic jack with a 2000 kN capacity, which was fixed to another reaction block. Both reaction blocks were fastened to the floor by high-strength steel rods, and two steel constraint rods were placed between the two reaction blocks to provide extra fixation. A load cell was placed between the jack and a steel cylinder within a sliding tube to record the axial forces, and to ensure that the steel cylinder was placed coaxially with the specimen, the steel cylinder was connected with a rigid support fixed to the floor. A rigid fixture, 200 mm in width, was mounted in mid-span of the specimen and connected to a hydraulic servo actuator with a capacity of 1000 kN to apply lateral cyclic loads. Moreover, to prevent the out-of-plane deformation of the specimen, two groups of lateral restriction device were used, and to minimize the effect of interface friction six ball bearings were placed on the specimen's sides. According to the size design in Fig. 3(a), the effective length of the specimen (L_e) was equal to 1660 mm.

The in-plane lateral displacements in mid-span and quarter points as well as the axial shortenings of the specimen were monitored by the displacement transducers (DTs), and each measuring point was equipped with two symmetrically distributed DTs. Moreover, a number of strain gauges (SGs) were mounted on the external wall of the outer and inner steel tubes to measure the development of longitudinal and transverse strains at the key section close to the fixture, as displayed in Fig. 3. Before commencing the tests, to guarantee the accuracy of the equipment and minimize potential experimental errors as much as possible, the measuring devices (DTs, SGs and load cell) and the loading equipment (actuator and hydraulic jack) were carefully calibrated. At the same time, a level

ruler was used to calibrate the horizontal coaxiality of the hydraulic jack, the hinges and the installed specimens, and a laser level meter was utilized to calibrate the verticality of the actuator and the vertical coaxiality of the actuator, the fixture and the mid-span section of the installed specimens.

2.4. Loading Protocols

During the tests, the axial compressive force was initially applied as per the pre-determined result and then remained unchanged, and the protocols for lateral cyclic loading presented in [33] were adopted in this study, as depicted in Fig. 4. In the elastic range, the tests were performed under force-control, and two cycles were imposed at the load levels of $0.25Q_c$, $0.5Q_c$ and $0.7Q_c$, where Q_c is lateral capacity estimated by the formulae for capacity of CFDST members [30]. In the subsequent inelastic range, the tests were done under displacement-control, and the displacement levels of $1.0\delta_y$, $1.5\delta_y$, $2.0\delta_y$, $3.0\delta_y$, $5.0\delta_y$, $7.0\delta_y$, $8.0\delta_y$ and $11.0\delta_y$ were applied to the specimen, where $\delta_y (= Q_c/K_{sec})$ is yield displacement, and K_{sec} is secant rigidity obtained by the first cycle of lateral load vs. mid-span lateral displacement hysteretic curves while $0.7Q_c$ is reached. Three cycles were imposed at the displacement levels of $1.0\delta_y$, $1.5\delta_y$ and $2.0\delta_y$, while two cycles were imposed for the remaining displacement levels.

3. Experimental Results and Discussion

3.1. Observed Damage Process and Failure Modes

It was observed that, the overall damage process of the FCFST specimens, indicating by the morphologic change of the outer tube at the control section, was analogous to that of the accompanying CFST specimen. Fig. 5 illustrates the representative damage process of specimen S30-4-0.05. Before reaching $0.7Q_c$, there was no apparent damage to the specimen, and during the $\pm 1.5\delta_y$ cycles the yielding of outer tube flanges was attained. With increasing lateral displacement to $\pm 2.0\delta_y$, slight outward local buckling (LB) appeared on the outer tube flanges; however, the LB disappeared when the lateral displacement reversed to zero. During the $\pm 3.0\delta_y$ cycles, the lateral load reached its peak, and simultaneously the LB of outer tube flanges became more severe. Upon further deforming, the lateral load began to decline and the outward LB of side walls in the outer tube

occurred during the $\pm 5.0\delta_y$ cycles. When the lateral displacement reached $\pm 7.0\delta_y$, the outward LB of flanges and side walls in the outer tube were connected to each other, resulting in the LB of corners in the outer tube. Finally, while during the $\pm 8.0\delta_y$ cycles, fracture occurred to the locally buckled corners in the outer tube.

The overall failure modes of the specimens after completing the tests are illustrated in Fig. 6. It should be noted that, the mid-span deflections of the specimens in the pictures were not pronounced, as the actuator was returned to the zero-load position to facilitate the removal of the fixture and the installation of the next specimen. For all the specimens, significant outward LB of walls and corners in the outer tube, either symmetrically or asymmetrically distributed in the vicinity of the fixture, were observed, and the LB form of the outer tube also shows some difference with the change of test variables. Similar phenomena were also observed in the previous tests of CFST and CFDST beam-columns under lateral cyclic loading [32, 34, 35]. This may be due to the drift of the actuator front end induced by the cyclic axial contraction of the specimens, as well as the random distribution of initial material and geometric imperfections of the specimens. Moreover, for the FCFSST specimens with a lower α_n and n as well as the reference CFST specimen, fracture of locally buckled corners occurs to the outer tube, considering that the plastic deformability of corners in the steel SHS is reduced after cold-forming process, resulting in the low-cycle fatigue destruction under cyclic loading.

Fig. 7 shows the typical failure modes of the sandwich concrete and inner tubes. It is shown that, in general, crushing and cracking coexist at the area corresponding to the outward LB of the outer tube, as alternating tension and compression happen to the sandwich concrete while subjected to cyclic loading, and under the combined compression and flexure, a number of longitudinal or transverse cracks also appear in the sandwich concrete away from the outward LB area of the outer tube. Meanwhile, χ and n are the key factors that determine the failure degree of the sandwich concrete, that is, with the increase of χ and n , the crushing/cracking degree of the sandwich concrete increased and accompanied by more and finer longitudinal cracks. For all the specimens, all the inner tubes produce the inward LB at the region corresponding to the outward LB of the outer

tube and crushing/cracking of the sandwich concrete, due to the restrictions of the sandwich concrete to their outward deformation, which is different from the interval outward and inward LB of the steel SHS/CHS [36, 37]. Generally, α_n has no obvious influence on failure modes of the inner tubes, while a larger χ and n leads to a severer inward LB of the inner tubes, and fracture of the inner tubes also occurs to the FCFSST specimens with $n \geq 0.25$ and $\chi = 0.5$, due to the increased initial damage under a higher axial compression and diameter/width-to-thickness ratio. Overall, the failure modes of the inner SHSs are severer than those of the inner CHSs as the former has a worse local stability than the latter. In addition, there are no obvious interface separation between different components and/or surface scratches on the sandwich concrete and steel tubes, meaning that the components in FCFSST specimens can generally work well together.

In summary, the failure of the outer tube, inner tubes and sandwich concrete of the FCFSST beam-column specimens under lateral cyclic loading occur sequentially, and gradually become severer with the increase of lateral displacement. The eventual failure modes of the specimens can generally be described as a combination of the following features, i.e., outward LB of outer tube (OLB_o), fracture following LB of outer tube (FLB_o), crushing/cracking of concrete (CC), inward LB of inner tube (ILB_i), and fracture following LB of inner tube (FLB_i), as schematically illustrated in Fig. 8 and listed in Table 1. It can be found that, the failure modes of the outer tube are largely determined by n and α_n , whilst the failure modes of the sandwich concrete and inner tubes depend more on χ and n . Avoiding or delaying local buckling of the steel tubes is the key target in the seismic design of the FCFSST members. Hence, the relevant variables should be rationally controlled in the design, among them, the most significant one is the axial compression level, followed by the width-to-thickness ratio of outer steel tube (equivalent to α_n), and then the width (diameter)-to-thickness ratio of inner steel tubes. The aim is to ensure that the outer steel tube providing confinement to the sandwich concrete does not undergo local buckling under earthquake action, or even if the tube wall does experience local buckling, it does not extend to the tube corners. Additionally, when the variables exceed the limits, considering setting stiffeners to the outer walls of the outer tube can improve the seismic

286 performance of such composite members.

287 3.2. Load Vs. Deformation Responses

288 Fig. 9 exhibits the measured lateral load (Q) vs. mid-span lateral displacement (δ) hysteretic curves,
289 where ‘OT’ and ‘IT’ are abbreviations for outer tube and inner tube, respectively. In addition, the
290 characteristic events, corresponding to points A to G on the $Q - \delta$ hysteretic curves, during the
291 whole destruction process under cyclic loading (typically shown in Fig. 5) are also clearly
292 demonstrated, where events A and C are identified using the measured longitudinal strains on the
293 outer and inner tubes, events B (including the corresponding load Q_{LB}), E, F and G are recognized
294 based on the observations of specimen appearance, and event D of reaching lateral capacity (Q_{ce})
295 corresponds to the peak load captured on the $Q - \delta$ hysteretic curves. It should be noted that, due to
296 the premature failure of the strain gauges on the inner tubes, event C of some specimens is not
297 captured. Generally, for the FCFSST specimens, before Q_{ce} is reached, events A~C occur
298 successively and LB of inner tube should have occurred but not visible, whilst after achieving Q_{ce} ,
299 due to the intensify of damage, events E and F appear sequentially and event G occurs to a small
300 number of the specimens. These results show that the capacity continues to improve after the yielding
301 and subsequent LB of the outer and inner tubes, meaning that Q_{ce} of the FCFSST specimens is
302 controlled by the properties of the sandwich concrete. However, for specimen C33-3-0.25, event B
303 occurs before event A, which is significantly different from other specimens, considering that the
304 width-to-thickness ratio of its outer tube (B_o/t_o) is obviously larger than the limit of 42.7 (i.e.,
305 $52\sqrt{235/f_{y0}}$) calculated according to [38], at which no LB occurs.

306 Overall, the measured $Q - \delta$ hysteretic curves are stabilized and do not show the characteristics
307 of pinching, which is similar to the results of previous tests on CFST and CFDST beam-columns [23,
308 24, 32, 35]. While α_n and n kept constant [Figs. 9(a)~(c) and Figs. 9(e), (i) and (k)], χ has a
309 moderate effect on the form of the $Q - \delta$ hysteretic curves, whilst both FCFSST specimens have
310 plumper $Q - \delta$ hysteretic curve than the reference CFST specimen, considering that the concrete
311 with a lower strength and ductility is replaced by the steel tubes with better mechanical properties.

For the FCFSST specimens under the same χ and α_n [Figs. 9(d)~(f) and Figs. 9(e), (i) and (k)], a higher n leads to a worse plumpness of the $Q - \delta$ hysteretic curve and a rapider capacity reduction after reaching Q_{ce} . This is due to the fact that, a severer initial damage of all components and a larger second-order effect is caused by a higher axial compression, which accelerates the failure process of the composite specimens under lateral cyclic loading. Moreover, when χ and n remain unaltered [Figs. 9(b), (e) and (g)], the FCFSST specimens with a larger α_n have a plumper $Q - \delta$ hysteretic curve, as the confinement received by the sandwich concrete from the outer tube enhances as α_n increases when the inner tubes provide reliable supports for the sandwich concrete [15, 23].

The recorded $Q - \delta$ backbone curves under different variables are shown in Fig. 10. The obtained lateral capacity (Q_{ce}) of the specimens, as listed in Table 1, is calculated as the average value of two peak loads. Generally, the trend of all $Q - \delta$ backbone curves in both loading directions is identical, and with increasing δ the load (Q) successively experiences approximate linear and subsequent nonlinear increase before reaching Q_{ce} and rapid decline after reaching Q_{ce} . Fig. 10(a) shows that, χ has little effect on the first two development phases and the third development phase in one loading direction of the $Q - \delta$ backbone curves, considering that the variation in χ of the specimens also leads to the change of the cross section, that is, the decrease in the area of sandwich concrete is accompanied by the increase in the area of inner tubes, and vice versa. Generally, the reduction in specimen performance caused by the decreased contribution of one material and the improvement in specimen performance caused by the increased contribution of the other material can counterbalance each other. While in the other loading direction, the load drop rate after reaching Q_{ce} when $\chi=0.50$ is evidently quicker than that when χ of 0.22 and 0.34. This may be due to the insufficient compaction of the sandwich concrete or the subtle discrepancy in position of the two inner tubes on the first compression side of the specimen, leading to faster crushing/cracking of the sandwich concrete and LB of the inner tubes. It can be seen from Fig. 10(b) that, n has a significant impact on the trend of the $Q - \delta$ backbone curves, and as n increases, the slope of the curves (i.e., tangential stiffness) before achieving Q_{ce} slightly increases; however, the lateral displacement corresponding

to Q_{ce} obviously decreases, and the load drop rate after reaching Q_{ce} becomes greater, due to the fact that the larger axial compression makes the sandwich concrete more dense, and the second-order effect speeds up the damage process of all the components, especially after reaching Q_{ce} . Additionally, when α_n is the same and χ and d_e are close, the FCFST specimens with inner SHSs have a slightly higher $Q - \delta$ backbone curve than those with inner CHSs, but the difference in the backbone curves is very minor, which differs from the marked disparity in the backbone curves exhibited by the CFDST beam-column specimens with outer CHS and SHS [23], as the confinement of the sandwich concrete by outer CHS is significantly superior to that by outer SHS. This can be explained by the fact that, although both the area and yield strength of the inner SHSs are slightly higher than those of the inner CHSs, the increase in the area of inner SHSs leads to the reduction in the area of sandwich concrete, thereby reducing the combined contribution of inner SHSs and sandwich concrete to the performance of the specimens. Fig. 10(c) indicates that, the higher α_n is, the larger Q under the same δ is, because the composite stiffness and the confinement to the sandwich concrete from the outer tube enhance with the increase of α_n [15, 23]. Generally, within the variable range of the present tests, the influence of α_n on the load drop rate after reaching Q_{ce} is moderate. As is shown in Fig. 10(d), both FCFST specimens have a larger slope of the curve (i.e., tangential stiffness) before achieving Q_{ce} and higher Q_{ce} together with the corresponding lateral displacement than the reference CFST specimen, meaning that replacing concrete core in the CFST is an effective scheme to improve the seismic performance, although the lateral load of the former drops slightly faster than that of the latter after Q_{ce} is reached.

The $Q - \varepsilon$ (strain) hysteretic curves of a typical specimen are displayed in Fig. 11, in which ‘L’ and ‘T’ in brackets stand for longitudinal and transverse strain respectively, and ε_{yo} (ε_{yi}) is yield strain of outer (inner) tube. Both longitudinal and transverse strains of the outer and inner tubes are relatively small before tube yielding, and all the measured strains are fully developed after yielding and subsequent LB of the tubes. For the outer tube, the strains at the flanges and near the corners are greater than those at the half-height section, as the measuring points are far away from the centroidal

axis, and generally the strains at the flanges are higher than those near the corners, because the LB occurs first to the flanges (Fig. 9). For the inner tubes, the strains at the circular arc vertex are lower than those at the flanges and near the corners of the outer tube as the measuring points are closer to the centroid axis; however, the transverse strains develop more fully than the longitudinal strains, which may be due to the deviation between the location of the measuring points and the LB position of the inner tubes.

Fig. 12 exhibits the effect of test variables on the $Q - \varepsilon$ hysteretic curves in a typical position (i.e., point ob). Generally, for the FCF SST specimens, a smaller χ and a larger α_n result in a more sufficient strain development, mainly due to a greater distance between the centroidal axis and the neutral axis of the cross-section and a better suppression of LB of the outer tube, respectively. Meanwhile, n has a moderate impact on the development of transverse strains; however, the lateral load vs. longitudinal strain hysteretic diagrams deviate to compressive side with increasing n , because a larger axial compression produces a higher initial compressive longitudinal strain. Moreover, the presence of the inner tubes does not cause significant difference in strain development between the FCF SST specimens and the reference CFST specimen.

The evolvement of axial shortening (u_a) of the specimens under different variables is presented in Fig. 13, where δ_{ye} is the measured yield displacement, which is defined below. Overall, the axial shortening (u_a) develops symmetrically while subjected to lateral cyclic displacements (δ), although there are certain fluctuations in the evolvement of u_a due to the randomness of the failure location and sequence as well as slight deviation of the testing set-up. In addition, the variation of u_a is limited before δ_{ye} is reached, while the u_a increases rapidly after achieving δ_{ye} . It can also be observed that, for the FCF SST specimens, χ has little influence on the evolvement of u_a , and when δ/δ_{ye} kept constant, the u_a and its gradient after reaching δ_{ye} increase as α_n decreases and n increases, which mainly causes the reduction of stiffness and the aggravation of damage, i.e., the weak ability to restrain axial shortening. In addition, due to the inner tubes' contribution to stiffness, the FCF SST specimens lead to slower early evolvement of u_a than the reference CFST specimen;

however, in the later stage, the difference in the evolvement of u_a between two types of composite specimens is gradually eliminated due to the gradual severity of the inner tubes' damage.

3.3 Lateral Capacity and Cross-sectional Flexural Stiffness

The columns in Fig. 14 demonstrate the variation in Q_{ce} . In the same condition, the Q_{ce} of the FCF SST specimens first goes up slightly and then goes down with the improvement in χ due to the inconsistency in the properties of the inner tubes, and reduces as n increases and α_n decreases due to the increased second-order effect and the decreased confinement of the outer tube to the sandwich concrete. In addition, the FCF SST specimens having inner CHSs result in slightly higher Q_{ce} than those having inner SHSs. On average, the ratio of Q_{ce} of the FCF SST specimens with χ of 0.34 and 0.50 to that of the composite specimen with $\chi=0.22$ is 1.02 and 0.96, respectively, the Q_{ce} of the FCF SST specimens having inner CHSs (SHSs) with n of 0.25 and 0.50 is 14.4% (5.5%) and 32.9% (26.5%) lower than that of the relevant FCF SST specimen with $n=0.05$, and the FCF SST specimens with α_n of 0.085 and 0.111 result in 15.3% and 29.8% higher Q_{ce} than their accompanying specimen with $\alpha_n=0.068$, respectively. Furthermore, while α_n and n kept constant, both FCF SST specimens have a higher Q_{ce} than the reference CFST specimen as part of concrete core is replaced by steel tubes with better mechanical properties, and the improvement percentage is 20.8%. Generally, the variation of Q_{LB} across specimens is similar to that of Q_{ce} .

With respect to the cross-section with strain gauges, the moment (M) vs. curvature (ϕ) backbone diagram of the specimens can be obtained according to the recorded $Q - \delta$ backbone curve, together with the measured extreme-fibre longitudinal strains. The formulae are expressed as:

$$M = Q/2 \cdot (L_e/2 - 60) + N_0 \cdot \delta \quad (4)$$

$$\phi = (\varepsilon_t - \varepsilon_c)/B_o \quad (5)$$

where, ε_c and ε_t are extreme-fibre compressive and tensile longitudinal strain measured at points oa and oe, respectively.

As a result, the cross-sectional flexural stiffness $[(EI)_e]$, defined as the ratio of $0.2M_{ce}$ to its corresponding curvature on the $M - \phi$ backbone diagram [39], is adopted to assess the ability to

resist the bending deformation of the specimens, in which M_{ce} is flexural capacity corresponding to Q_{ce} , and the values of the obtained $(EI)_e$ of the specimens are given in Table 1 and further comparatively illustrated in Fig. 14 using diamond points. Overall, the $(EI)_e$ of the FCFSSST specimens reduces with the augment of χ , but the reduction is not very high, that is, the $(EI)_e$ for $\chi=0.34$ and $\chi=0.50$ is 2.2% and 4.2% lower than that for $\chi=0.22$, respectively. It should be noted that, the $(EI)_e$ under varied χ mainly depends on the geometry of the inner tubes. The $(EI)_e$ of the FCFSSST specimens enlarges as α_n and n increase, which is consistent with previous experimental findings of CFST and CFDST beam-columns under lateral cyclic loading [23, 32]. This is due to the fact that the moment of inertia of the outer tube increases with the increment of α_n , and the increment of axial compression can improve the compaction of the sandwich concrete, and thus reduce the effect of the inherent defects of concrete. Simultaneously, consistent with the pattern embodied in the Q_{ce} , the FCFSSST specimens having inner CHSs possess slightly higher $(EI)_e$ than those having inner SHSs. The $(EI)_e$ of the FCFSSST specimens with $\alpha_n=0.085$ and $\alpha_n=0.111$ is 8.8% and 27.5% higher than that of the relevant FCFSSST specimen with $\alpha_n=0.068$, and the $(EI)_e$ of the FCFSSST specimens having inner CHSs (SHSs) with n of 0.25 and 0.50 is 5.7% (2.9%) and 9.4% (6.0%) higher than that of the FCFSSST specimen with $n=0.05$, respectively. It can be concluded that α_n is the variable that notably affects the cross-sectional flexural stiffness of the FCFSSST beam-columns. In addition, the two FCFSSST specimens respectively produce 9.5% and 5.0% higher $(EI)_e$ than the reference CFST specimen, meaning that replacing partial concrete with multiple steel tubes can also improve the cross-sectional flexural stiffness of the FCFSSST beam-columns.

3.4 Stiffness Degradation Based on $Q - \delta$ Hysteretic Curves

According to the Chinese specification for seismic test [40], the stiffness degradation of the specimens under lateral cyclic loading can be evaluated by deterioration of secant stiffness from the measured $Q - \delta$ hysteretic curves. The secant stiffness during each cycle (K_i) can be expressed as follows:

$$K_i = (|-Q_i| + |+Q_i|)/(|-\delta_i| + |+\delta_i|) \quad (6)$$

where, $+Q_i$ and $-Q_i$ are positive and negative peak lateral load of i th loop, and $+\delta_i$ and $-\delta_i$ are

the corresponding lateral displacement. Only the first cycle of each displacement amplitude is considered in the calculation.

The stiffness degradation of the specimens described by the relationship between (K_i/K_1) and δ/δ_{ye} is presented in Fig. 15. As shown in the figures, when lateral displacement gradually increases, the stiffness of the specimens rapidly decreases due to the emergence and gradual accumulation of concrete damage (mainly tensile cracking and crushing), as well as the subsequent gradually aggravated buckling of the outer and inner tubes caused by the weakening of the supporting effect of the sandwich concrete on the steel tubes and the interfacial damage between different components. When δ/δ_{ye} reaches 4.0, the remaining stiffness is approximately 16.8% to 26.8% of K_1 , except for two FCFSSST specimens with $n=0.5$. In the later stage, the completely damaged sandwich concrete can no longer provide support for the inner and outer tubes, and only the effective area of the steel tubes can offer stiffness for the specimens, and the remaining stiffness is only about 7% to 10% of K_1 . Meanwhile, there is no inflection point on the $K_i/K_1 - \delta/\delta_{ye}$ curves, indicating that the FCFSSST beam-columns possess a stable seismic performance.

Generally, for the FCFSSST specimens, χ has a moderate influence on the stiffness degradation. The $K_i/K_1 - \delta/\delta_{ye}$ curves are closely aligned under different void ratios, which is in line with the $Q - \delta$ backbone curves as depicted in Fig. 15(a). The curves in Figs. 15(b) and (c) suggest that a larger n and a lower α_n result in a faster stiffness degradation due to the increased damage of the sandwich concrete under a higher axial compression and the decreased confinement of the outer tube to the sandwich concrete, respectively. Particularly, as shown in Fig. 15(b), the FCFSSST specimens with inner SHSs experience a faster stiffness degradation than those with inner CHSs. In this study, the inner SHSs have poorer local stability than the inner CHSs, which weakens the supporting effect of the inner tubes on the deformation of the sandwich concrete. It can be observed from Fig. 15(d) that before δ/δ_{ye} reaches 2.0, the stiffness degradation of the reference CFST specimen lies between that of the two FCFSSST specimens. However, when $\delta/\delta_{ye} > 2.0$, the stiffness degradation of the former is slower than that of the latter, which might be attributed to the LB of the inner tubes in

the latter, leading to a weaker support for the sandwich concrete.

3.5 Ductility Coefficient and Ultimate Drift

To assess the ductility of the specimens, the ductility coefficient (μ) related to the key lateral displacements is defined as follows:

$$\mu = \delta_{ue} / \delta_{ye} \quad (7)$$

where, δ_{ue} is ultimate displacement, which equals to lateral displacement when the load in the post-peak stage drops to $0.85Q_{ce}$. In this study, the method in [41] was adopted to determine the value of yield displacement (δ_{ye}), as shown in Fig. 16, that is, the horizontal coordinate of the straight line passing through the starting point and the measured data point with vertical coordinate of $0.7Q_{ce}$ when the vertical coordinate equals to Q_{ce} .

The ultimate drift (θ_u), which was considered as an indicator that reflects the deformability of the specimens [42, 43], is also acquired in this study, and the formula is as follows:

$$\theta_u = 2\delta_{ue} / L_e \quad (8)$$

The average values of μ and θ_u for each specimen are summarized in Table 1, and the effect of variables on them are depicted in Fig. 17. It can be observed that, the results of μ and θ_u indicate similar evolutionary characteristics with the change of variables, and in this study the variable n is the most important influencing factor. The μ and θ_u of the FCFSSST specimens are negatively correlated with the variables χ and n , which may be caused by the more serious damage of the sandwich concrete under the increased LB deformation of the inner tubes and second-order effect in the post-peak stage. Compared to the case of $\chi=0.22$, the $\mu(\theta_u)$ for the cases of $\chi=0.34$ and $\chi=0.50$ are reduced by 13.3%(3.7%) and 15.0%(11.3%), respectively. Similar to the CFDSTs with inner CHS in [23], the μ and θ_u of the FCFSSST specimens with inner CHSs are slightly larger than those of the ones with inner SHSs. For the specimens with CHSs, the $\mu(\theta_u)$ for the cases of $n=0.25$ and $n=0.50$ are 35.6% (43.8%) and 60.9% (70.4%) lower than those for the case of $n=0.05$, respectively. In contrast, for the specimens with SHSs, the reductions are 34.4% (49.9%) and 67.7% (76.4%), respectively. Although replacing a single inner tube with multiple inner tubes mitigates the adverse

effects of the inner steel SHS, the discrepancy still persists. The μ and θ_u of the FCFSSST specimens are positively correlated with the variable α_n due to the increased confinement of the outer tube to the sandwich concrete, and the improvement in $\mu(\theta_u)$ for the cases of $\alpha_n=0.085$ and $\alpha_n=0.111$ is 11.3% (6.7%) and 15.4% (14.8%) compared to the case of $\alpha_n=0.068$, respectively. Moreover, with respect to the reference CFST specimen, both FCFSSST specimens produce lower μ and θ_u , and when the inner tubes are steel CHSs (SHSs) the reduction percentage is 22.7% (25.2%) and 8.2% (20.1%), respectively. This may be due to the fact that, in the post-peak phase, the sandwich concrete deforms inward while the serious LB happens to the inner tubes, thus accelerating destruction of the FCFSSST specimens, while the concrete core in the reference CFST specimen does not produce the inward deformation.

According to ANSI/AISC 314-16 [31], moderately ductile members are expected to have moderate plastic rotation of 0.02 rad or less, while highly ductile members are intended to have significant plastic rotation of 0.04. Although the ductility of FCFSSST specimens is somewhat lower than that of the reference CFST specimen, they still meet the requirements for highly ductile members, except for two specimens under a higher axial compression level ($n=0.5$). The θ_u of specimen C33-4-0.50 is 2.42, which meets the requirement of moderate ductility, while the θ_u of the specimen S30-4-0.50 is 1.89, which is lower than the corresponding rotation of moderate ductility. As a result, when the FCFSSST members are utilized in seismic-resistant structures, their axial compression level should be strictly controlled and not exceed 0.5, and simultaneously, it is advisable to use steel CHS for the inner tubes.

3.6. Energy Dissipation

The accumulated energy dissipation of the specimens (E_a) can be acquired by summing the area within $Q - \delta$ hysteretic diagram [40]. The specific values of accumulated energy dissipation ($E_{a,s}$), corresponding to the load dropping to $0.85Q_{ce}$, are provided in Table 1. The effect of variables on the relationship between E_a and δ/δ_{ye} of the specimens is shown in Fig. 18. The dissipated energy is negligible when δ is less than δ_{ye} , that is, the energy dissipation ability of the specimens is mainly

dependent on their post-yield performance. Generally, under the same δ/δ_{ye} , the E_a of the FCFSSST specimens rises as α_n increases and n decreases. This is because the improvement in the confinement of outer tube and the reduction in the second-order effect result in the simultaneous increase of lateral capacity (Fig. 14) and ductility [Figs. 17(b) and (c)]. When χ increases from 0.22 to 0.50, the E_a of the FCFSSST specimens demonstrates a trend of initially decreasing and then increasing, which reflects the variation of lateral capacity but does not align with the change of ductility coefficient. Simultaneously, the difference in the energy dissipation characteristics between the specimens with inner CHSs and those with inner SHSs is inconsistent when n varies. These circumstances indicate that further experimental studies on the influence rule of χ and the type of inner tube are necessary. Additionally, both FCFSSST specimens end up with a higher E_a than their counterpart CFST specimen, mainly due to the replacement of part of concrete core with ductile steel tubes. Specifically, for the FCFSSST specimens, the $E_{a,s}$ at α_n of 0.085 and 0.111 is 28.4% and 56.9% higher than that at $\alpha_n=0.068$, and the $E_{a,s}$ at n of 0.25 and 0.05 is 3.0-10.8 times (with inner CHSs) and 3.0-9.5 times (with inner SHSs) of that at $n=0.50$. Meanwhile, the $E_{a,s}$ of the FCFSSST specimens is 11.7% (with inner CHSs) and 21.7% (with inner SHSs) higher than that of the reference CFST specimen.

4. Prediction of Mechanical Indexes

Currently, there is no practical method for the calculation of mechanical indexes of the FCFSSST compression-flexure members. Therefore, the simplified methods of CFST and CFDST members in the existing design specifications [30, 38, 44] are tentatively used to try to develop the calculation method for the mechanical indexes of the FCFSSST beam-columns under lateral cyclic loading.

4.1. Flexural Capacity

The comparison between axial compression (N) vs. moment (M) correlation curves based on the existing design specifications and the measured capacities of the FCFSSST beam-column specimens in this study is demonstrated in Fig. 19, in which all three approaches in [44] that are differentiated based on B_o/t_o are included. Simultaneously, the predicted flexural capacities (M_{cc}) and their

comparison to the measured ones (M_{ce}) are presented in Table 4, where the results based on approaches (2) and (3) in [44] are presented, while the results of approach (1) without considering the contribution of concrete are ignored. In the calculation, all the material partial safety factors are set to unity. The results show that, the approaches in three design specifications all underestimate the flexural capacity of the FCFSST beam-column specimens, although EC 4 [38] and AISC [44] yield better predictions than T/CCES [30]. It is believed that, the conservative predictions are mainly due to the neglect of the strengthening effect of steel under cyclic loading [45, 46].

The previous studies on the stress vs. strain backbone curve under cyclic loading and the corresponding stress vs. strain curve under monotonic loading of different kinds of steel [45, 46] showed that, the strength of steel under cyclic loading was improved by about 20%. An experimental database, integrating extremely low cycle fatigue test data from previous researchers [45, 47-50], has been established to further validate the value of this amplification factor. The relationship between the steel yield strength under cyclic and monotonic loading from the tests is depicted in Fig. 20, where, f_y represents the steel yield strength under monotonic loading (tension), and f_y^c is the steel yield strength under cyclic loading. Overall, the ratio of f_y^c to f_y ranges from 1.0 to 1.4, with the majority of data points clustering around 1.2. Therefore, the measured results of f_{yo} and f_{yi} in Table 2 are multiplied by an amplification factor of 1.2 to take into account the influence of cyclic loading, and the recalculated $N - M$ correlation curve and capacity of the FCFSST beam-column specimens, presented with a supplementary capital letter 'M', are also presented in Fig. 19 and Table 4, respectively. It can be observed that, generally, the modified method in this study can predict the $N - M$ correlation curve and flexural capacity of FCFSST beam-columns more accurately; however, the predicted results of EC 4 and AISC are slightly higher than the measured ones, that is, the mean value and standard deviation (SD) of the ratio of $M_{cc,M}$ to M_{ce} are 1.011 and 0.031 respectively, while T/CCES provides more accurate and safe predictions, that is, the mean value and SD of the ratio of $M_{cc,M}$ to M_{ce} are 0.923 and 0.045, respectively. It is of vital importance to recognize that the extent of cyclic hardening in steel is influenced by multiple factors, such as the steel grade and strain

amplitudes. The amplification factor introduced in this research serves as an initial approximation for estimating the flexural capacity of the FCFSSST beam-columns under cyclic loading. For future specification applications, more experimental and numerical investigations should be conducted to obtain the accurate amplification factor of steel yield strength under cyclic loading.

4.2. Cross-sectional Flexural Stiffness

The cross-sectional flexural stiffness of composite sections in the aforementioned design specifications is calculated using the superposition method, while taking into account the reduction caused by concrete cracking, and the equation can be uniformly written as the following:

$$K_{ic} = E_{so} \cdot I_{so} + E_{si} \cdot I_{si} + \beta_c \cdot E_c \cdot I_c \quad (9)$$

where, E_{so} and E_{si} are elastic modulus of the outer and inner steel tubes respectively, I_{so} , I_{si} and I_c are moment of inertia of the outer steel tube, the inner steel tubes and the un-cracked sandwich concrete, respectively, and β_c is reduction coefficient. In T/CCES [30] and EC 4 [38], the value of β_c is set as 0.6 for square composite sections, while in AISC [44], the value of β_c equals to $0.45 + 3A_s/A_g$ and must be less than or equal to 0.9, in which A_s and A_g are cross-sectional area of the outer steel tube and gross area of composite section, respectively.

The calculated values of cross-sectional flexural stiffness $[(EI)_c]$ of the FCFSSST specimens are compared with the measured results $[(EI)_e]$ in Table 5. The comparison shows that, the AISC gives the best predictions, and the mean value of $(EI)_c/(EI)_e$ is 0.976 with a SD of 0.038. Overall, the results of $(EI)_c$ by the EC4 and T/CCES are somewhat conservative, and the mean value and SD of $(EI)_c/(EI)_e$ are 0.892 and 0.034, respectively. As previously mentioned, the impact of the axial compression level on the measured stiffness of the FCFSSST beam-columns is not negligible, but it is not adequately considered in the current design specifications, resulting in conservative predictions. Additionally, both EC4 and T/CCES assign a fixed value of 0.6 to the β_c for all square composite sections, disregarding the fact that different thicknesses of steel tubes lead to varying levels of confinement/support to the sandwich concrete, which in turn affects the cross-sectional flexural stiffness of the sandwich concrete, thus leading to more conservative predictions compared to AISC.

A more rational and precise method for calculating β_c , which takes into account the composite action between the steel tubes and sandwich concrete and the influence of the axial compression level, could be derived through further numerical analysis under a broader range of variables.

5. Conclusions

The seismic performance of four-opening concrete filled sandwich steel tube (FCFSST) beam-columns is studied via pseudo-static tests of typical specimens, and several important conclusions are as following:

(1) While subjected to lateral cyclic loading, the damage process of the FCFSST specimens closely resembles that of the relevant CFST specimen. The dominating failure modes of the FCFSST specimens are characterized by a combination of outward LB and fracture following LB of outer tube, crushing/cracking of sandwich concrete and inward LB and fracture following LB of inner tubes. The failure modes of the outer tube depend more on n and α_n , and those of the sandwich concrete are mainly determined by χ and n , while those of the inner tubes are mostly governed by χ and their sectional type.

(2) Generally, the load vs. deformation hysteretic curve of the FCFSST specimens is stabilized and does not exhibit the properties of pinching. For the FCFSST specimens, a lower n and a larger α_n result in a better plumpness of the $Q - \delta$ hysteretic curve, a slower stiffness degradation and a higher energy dissipation. Accordingly, for the $Q - \delta$ backbone curves, n has the greatest effect, followed by α_n , while χ and sectional type of inner tubes have moderate influence. In addition, the FCFSST specimens have a plumper $Q - \delta$ hysteretic curve, a quicker stiffness degradation when $\delta/\delta_{ye} > 2.0$ and a higher energy dissipation than the reference CFST specimen.

(3) For the FCFSST specimens, with the increase of χ , the Q_{ce} first increases mildly and then decreases, and the $(EI)_e$, μ and θ_u decrease, and the peak reduction of them is 4.0%, 4.2%, 15.0% and 11.3%, respectively. As n enlarges, the Q_{ce} , μ and θ_u reduces and the $(EI)_e$ improves, and the peak reduction of the first three indicators is 32.9%, 67.7% and 76.4%, respectively, and the peak increase of the last indicator is 9.4%. The Q_{ce} , $(EI)_e$, μ and θ_u are positively correlated with α_n ,

and the peak improvement corresponding to the maximum α_n is 29.8%, 27.5%, 15.4% and 14.8%, respectively. Moreover, the FCF SST specimens have higher Q_{ce} and $(EI)_e$, but lower μ and θ_u than the accompanying CFST specimen. It is suggested that the axial compression level of the FCF SST beam-columns in seismic-resistant structures should not exceed 0.5. Simultaneously, it is more advisable to use steel CHSs for the inner tubes from the perspective of enhancing ductility.

(4) The flexural capacity of the FCF SST beam-columns can be accurately and safely predicted by the approach in T/CCES after considering the increased steel strength under cyclic loading. Moreover, AISC provides a more accurate prediction for the cross-sectional flexural stiffness of the FCF SST specimens than the EC4 and T/CCES.

It is worthy noting that more typical tests (e.g., considering the effects of different concrete mixes) and a detailed numerical analysis under a wider range of variables shall be studied in the future to better reveal the working mechanism of FCF SST beam-columns under cyclic loading and provide a data basis for proposing a more accurate design method as well as a reasonable restoring force model. Furthermore, potential strategies to enhance the seismic performance of the FCF SST beam-columns, such as measures aimed at improving ductility and energy dissipation capacity, should be explored through experimental and/or numerical studies.

CRedit authorship contribution statement

Dong-Jie Zhang: Methodology, Data curation, Visualization, Writing - original draft, Writing - review & editing, Investigation. **You-Fu Yang:** Conceptualization, Methodology, Supervision, Project administration, Funding acquisition, Writing - review & editing. **Feng Fu:** Writing - review & editing, Methodology, Conceptualization.

Declaration of Competing Interest

The authors declared that there is no conflict of interest.

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Experimental behaviour of four-opening concrete filled sandwich steel tube (FCFSST) beam-columns under lateral cyclic loading

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Abstract: In this paper, a new type of composite sections, which are composed of one outer square steel tube, four inner circular or square steel tubes and filled concrete, four-opening concrete filled sandwich steel tube (FCFSST), is first developed. To assess seismic response of typical beam-columns, the tests under axial compression and lateral cyclic loading were conducted on ten FCFSST specimens and one counterpart concrete filled steel tube (CFST) specimen. Effect of several key variables, including void ratio (χ), axial compression level (n), nominal cross-sectional steel ratio (α_n) and sectional type of inner tubes, on the seismic performance of representative FCFSST specimens was investigated in detail. The experimental observations show that, the failure modes of FCFSST beam-column specimens include outward local buckling (LB) and fracture following LB of outer tube, inward LB and fracture following LB of inner tubes and crushing/cracking of sandwich concrete, and generally n and α_n are the controlling factor for the failure of outer tube, whilst χ and sectional type mainly affect the failure of inner tubes. For the FCFSST specimens, a lower n and a larger α_n result in a superior plumpness of the hysteretic curve, a slower stiffness degradation, a higher energy dissipation, and a better ductility, and the influence of n and α_n on key factors is more significant than that of χ . Additionally, in comparison with the reference CFST specimen, the FCFSST specimens show a 20.8% increase in lateral capacity, a 5.0-9.5% increase in cross-sectional flexural stiffness, and a 11.7-21.7% increase in energy dissipation, while presenting a 22.7-25.2% reduction in ductility coefficient and an 8.2-20.1% reduction in ultimate drift, respectively. Finally, the approaches in the existing design specifications of composite structures were used to predict

flexural capacity and cross-sectional flexural stiffness of FCFSSST beam-columns under lateral cyclic loading, and the results indicate that the approach in T/CCES can precisely and safely predict the flexural capacity by considering the increased steel yield strength under cyclic loading, whereas the approach in AISC can offer an accurate prediction for the cross-sectional flexural stiffness.

Keywords: Four-opening concrete filled sandwich steel tube (FCFSST); Beam-columns; Lateral cyclic loading; Capacity; Cross-sectional flexural stiffness; Design specification.

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1. Introduction

There have been many applications of concrete filled steel tube (CFST) in practical engineering [1], considering that the structural responses and design methods of CFST structures have been comprehensively recognized in the past decades [2-7]. Nowadays, due to the increased performance requirements, various improved CFST sections have been developed and investigated systematically, including the application of new materials and modified cross-sectional designs, for instance, the CFST members using ultra-high-strength steel tube and concrete infill [8-10] to minimize size and increase elastic deformability, the double-skin CFST (i.e., CFDST) [11-13] and double-opening concrete filled sandwich steel tube (DCFSST) configurations [14, 15] to reduce self-weight and enhance fire resistance, and fibre-reinforced polymer (FRP) confined concrete [16, 17] or CFST [18, 19] to improve corrosion resistance and/or resistance of thin-walled steel tube to local buckling. The research outcomes [8-15, 18-19] indicate that, compared to the traditional CFST, the improved CFSTs have similar or better structural performance, but their analytical theory and design approach need to be upgraded with the inclusion of some parameters related to the modified characteristics.

A new type of improved CFST, four-opening concrete filled sandwich steel tube (FCFSST), is developed in this study. The FCFSSST member is produced by replacing a portion of concrete core in traditional square CFST member with four symmetrically distributed steel circular hollow sections (CHSs) or square hollow sections (SHSs). It can be judged that, this new type of innovative cross-

section form offers a larger cross-sectional flexural stiffness, a higher bending resistance, a lower self-weight and a better fire resistance than the traditional CFST. While compared to the CFDST, the FCFSSST has better flexural performance as its inner tubes are far away from the sectional centroidal axis, and replacing one large-sized inner tube by four small-sized inner tubes enables the modular assembly, thereby achieving cost reduction in transporting and processing of the inner tubes. Simultaneously, the FCFSSST has identical performance in both loading directions, which makes it have a wider range of engineering applications than rectangular DCFSSST. It is anticipated that, similar to rectangular DCFSSST [14, 15], the FCFSSST can be utilized as new structural members in high seismic activity regions to withstand extreme loads and contribute high stiffness and stability, such as support columns for offshore platforms, supporting columns on subway platforms, tunnel supports in urban rail transit projects, giant columns for super high-rise buildings, and tower columns for bridges or space structures. The fabrication of new FCFSSST mainly includes: connecting the inner tubes first, then connecting the outer tube, and finally casting concrete in sections or as a whole. To facilitate the connection of the segmented steel tubes, the free surface of the inner tubes should be larger than that of the outer tube. Additionally, the FCFSSST can be regarded as a new reinforcement technology that employs large-sized steel tubes and filled concrete to retrofit or strengthen existing small-sized tubular members/structures (e.g., lattice columns in factory buildings, pillars in transmission structures, arch ribs in arch bridges), with the aim of effectively enhancing the structural performance and the ability to resist harsh conditions. Before the reinforcement construction, each segment of the new outer tube needs to be divided into two U- or L-shaped parts. The reinforcement sequence is usually to position one part of the outer tube segment and temporarily fix it to the existing tubular members/structures first, then align the other part of the outer tube segment with the fixed part, and finally pour concrete in sections or as a whole.

Currently, there is little research of evaluating structural behaviour of FCFSSST members, and the existing studies are limited to the static properties of the improved CFST members with similar composition to the FCFSSST. Pi et al. [20] and Guo et al. [21] conducted experimental investigation

into the structural performance of axially compressed CFST short columns with double steel SHSs/CHSs, and further proposed strength calculation formulae for the above three types of composite members. Xu [22] carried out finite element (FE) modelling on axial compressive property of the FCFSSST short columns with outer and inner steel CHSs. Our research group [14, 15] performed experimental study and FE modelling on rectangular DCFSSST short columns under axial compression and beams under bending, and finally suggested the simplified calculation method for their load-carrying capacity.

As it is widely known, when structural members or structures are located in an earthquake zone, it is necessary to conduct reliable seismic performance evaluation and effective seismic design for them. Significant research on the seismic performance of CFDST members has been conducted over the past 20 years. The pseudo-static test, a prevalent method in this field, has been widely adopted to assess the seismic performance of circular-in-circular CFDST beam-columns [23], rectangular-in-rectangular CFDST box piers [24, 25], square-in-square CFDST box columns stiffened with PBL [26], and engineered cementitious composite (ECC) filled high-strength CFDST columns with outer and inner steel CHSs [27]. Furthermore, shaking table tests have been reported on the dynamic behaviour of post-tensioned precast circular-in-circular CFDST piers under tri-directional ground motions [28]. The numerical simulations subsequent to the experimental study have also been carried out to facilitate a deeper comprehension of the seismic performance of CFDST members. For instance, Xia et al. [25] presented a FE model for double-skin steel-concrete composite box (DSCB) piers under lateral cyclic loads, considering the local buckling of steel tubes, and Okamoto and Nakamura [29] conducted seismic analysis of square-in-square CFDST towers in multi-span cable-stayed bridges based on the fiber model method. However, there are no reports on the seismic behaviour of FCFSSST members, which hinders their application in earthquake-prone regions.

The novelty of this work lies in the development of a new type of composite member originated from square CFST, namely FCFSSST. Presented here are the outcomes of an experiment study on the seismic performance and calculation method for mechanical indexes of the proposed composite

beam-columns. From the tests, the typical specimens demonstrate excellent seismic performance. Also, to develop a design method, the axial compression vs. moment correlation curves for CFST and CFDST members are inspected to explore the applicability of them in predicting the flexural capacity of the proposed FCFSSST beam-columns. Moreover, previous studies on the performance of other thematically similar composite members [14, 15, 23-25, 28] offer valuable insights and references for this work, especially regarding the working mechanism and failure modes of different components.

2. Experimental Investigation

2.1. Specimen Design

The objective of this experiment is to evaluate the influence of key variables on the seismic performance of FCFSSST beam-columns and to provide model test data for verifying the accuracy of the design method of mechanical indexes. Eleven composite beam-columns, including 7 FCFSSST specimens containing inner CHSs, 3 FCFSSST specimens containing inner SHSs and one companion CFST specimen, were prepared for the pseudo-static tests under axial compression and lateral cyclic loading. The length of the specimens (L), including the two endplates with a thickness of 20 mm, was 1600 mm. The cross-section of the specimens is depicted in Fig. 1, where B_o is overall width of outer SHS, D_i is overall width (diameter) of inner SHSs (CHSs), t_o (t_i) is wall thickness of outer (inner) tube, and d_e is distance between centroid of two inner tubes.

The key variables in the tests included void ratio (χ), axial compression level (n) and nominal cross-sectional steel ratio (α_n), and the detailed expressions are as follows:

$$\chi = \sqrt{\sum_{i=1}^4 A_{it,i} / A_{ce}} \quad (1)$$

where, $A_{it,i}$ is cross-sectional area within exterior wall of the i th inner tube, and A_{ce} is cross-sectional area within internal wall of outer tube.

$$n = N_0 / N_{cr} \quad (2)$$

where, N_0 is unaltered axial compressive force applied to the specimens, and N_{cr} is stable bearing capacity of the specimens calculated using the formulae for the CFDST and CFST columns in [30].

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$$\alpha_n = A_{ot}/A_{ce} \quad (3)$$

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where, A_{ot} is cross-sectional area of outer tube.

The specimens in this study were scaled columns used in a subway station platform with a scale ratio of about 1:5, and were designed in accordance to relevant specifications, namely T/CCES 7-2020[30] and AISC 341-16[31], and previous research [23]. It is well known that the seismic performance of structural members is significantly affected by load, material and geometric parameters. In this study, the selected variable n reflects the change in load, while the selected variable α_n exhibits the change in material and geometric dimensions. The ranges of n (0.05-0.50) and α_n (0.068-0.111) generally correspond to the typical column configuration and seismic design strategy in engineering practice [23, 30, 31]. The selection of specific materials was based on T/CCES 7-2020[30] and took into account the capacity of testing devices. Furthermore, considering the cross-section form of the FCFST, the void ratio (χ) is a potentially significant factor affecting its seismic performance. According to the limit for the void ratio of CFDST in [30], a sufficient gap between inner and outer steel tubes is required to ensure the pouring quality and mechanical performance of concrete, and the maximum void ratio of the specimens was finally determined and set as 0.50, with a variation range of 0.22 to 0.50.

The details of the specimens are summarized in Table 1, where $f_{yo}(f_{yi})$ is yield strength of outer (inner) steel tube, Q_{LB} is initiation load of local buckling of outer tube, Q_{ce} is lateral capacity, $(EI)_e$ is cross-sectional flexural stiffness, μ is ductility coefficient, θ_u is ultimate drift, and $E_{a,s}$ is accumulated energy dissipation when the load drops to $0.85Q_{ce}$. In addition, each specimen is assigned an ID, starting with the type of cross-section, where ‘C’ stands for the FCFST specimen having inner CHSs, ‘S’ stands for the FCFST specimen having inner SHSs and ‘Ref’ stands for the reference CFST specimen, together with overall width or diameter of inner tubes, followed by the nominal wall thickness of outer tube, and ending with the axial compression level.

2.2. Material Properties and Specimen Production

Seven different types of steel tubes were used to make the outer and inner tubes of the specimens.

Three standard coupons were tested under axial tension to obtain the properties of the steel tube, which were derived from the average of the measured results, as presented in Fig. 2 and Table 2. A commercially ready-mixed concrete with a design strength grade of C40 was adopted to fill the gaps between the inner and outer steel tubes. The concrete mixture was prepared with the following materials as specified by the supplier: Ordinary Portland Cement (P.O 42.5R), crushed granite coarse aggregate with a maximum size of 20 mm, river sand, fly ash (Grade I), tap water, and polycarboxylic acid series of high-performance water reducer. The compressive strength of concrete after curing for 28 days ($f_{cu,0}$) and during cyclic tests (f_{cu}) of composite specimens were measured by a sufficient number of cubes having sides of 150 mm. The axial compressive tests of three prismatic blocks were conducted to obtain elastic modulus of concrete (E_c). The slump and spread were also recorded before casting the concrete. The mixture proportion and properties of concrete is presented in Table 3, where f_{co} is the characteristic compressive strength converted from f_{cu} according to Eurocode.

The 'direct square forming' process was employed to fabricate the outer tubes. In this process, a hot rolled steel plate with length of 1600 mm was directly bent into a square tube at room temperature, and the final cold-formed steel SHSs consisted of a direct butt weld located in the middle of the side width. Meanwhile, the finished cold-formed steel SHSs or CHSs were used as the inner tubes. There were two endplates for each specimen, one of which was machined with four holes adapted to the outer diameter of the inner tubes, and the edge of the holes was beveled to facilitate reliable welding between the endplate and the inner tubes. Before placing the sandwich concrete, the endplate without holes was sequentially welded to the inner tubes and the outer tube using the fillet welds. The curing of the sandwich concrete was accomplished in an outdoor open space. To maintain the moist state during the curing period, the exposed portion of the sandwich concrete on the top of the specimens was covered with cotton cloth and watered regularly. After curing for 14 days, the sandwich concrete was smoothed against the upper surface of outer tube, and the endplate with four holes was welded to both inner and outer tubes. Finally, to prevent the occurrence of end destruction during the loading process, eight stiffeners were welded to each endplate and the outer steel SHS simultaneously.

2.3. Testing Set-up and Instrumentation

The composite specimens in this study were designed to replicate the frame columns between the inflection points of adjacent floors, and the inter-story shear induced by an earthquake was transferred from the rigid floor to the column at the floor level [32]. As depicted in Fig. 3, the composite specimen was horizontally arranged with both endplates connected to a hinge through high-strength bolts. One hinge together with two pin plates were fixed by connecting with a reaction block, and the other hinge together with two pin plates were connected to a set of devices for applying axial forces. The constant axial compression was imposed using a hydraulic jack with a 2000 kN capacity, which was fixed to another reaction block. Both reaction blocks were fastened to the floor by high-strength steel rods, and two steel constraint rods were placed between the two reaction blocks to provide extra fixation. A load cell was placed between the jack and a steel cylinder within a sliding tube to record the axial forces, and to ensure that the steel cylinder was placed coaxially with the specimen, the steel cylinder was connected with a rigid support fixed to the floor. A rigid fixture, 200 mm in width, was mounted in mid-span of the specimen and connected to a hydraulic servo actuator with a capacity of 1000 kN to apply lateral cyclic loads. Moreover, to prevent the out-of-plane deformation of the specimen, two groups of lateral restriction device were used, and to minimize the effect of interface friction six ball bearings were placed on the specimen's sides. According to the size design in Fig. 3(a), the effective length of the specimen (L_e) was equal to 1660 mm.

The in-plane lateral displacements in mid-span and quarter points as well as the axial shortenings of the specimen were monitored by the displacement transducers (DTs), and each measuring point was equipped with two symmetrically distributed DTs. Moreover, a number of strain gauges (SGs) were mounted on the external wall of the outer and inner steel tubes to measure the development of longitudinal and transverse strains at the key section close to the fixture, as displayed in Fig. 3. Before commencing the tests, to guarantee the accuracy of the equipment and minimize potential experimental errors as much as possible, the measuring devices (DTs, SGs and load cell) and the loading equipment (actuator and hydraulic jack) were carefully calibrated. At the same time, a level

ruler was used to calibrate the horizontal coaxiality of the hydraulic jack, the hinges and the installed specimens, and a laser level meter was utilized to calibrate the verticality of the actuator and the vertical coaxiality of the actuator, the fixture and the mid-span section of the installed specimens.

2.4. Loading Protocols

During the tests, the axial compressive force was initially applied as per the pre-determined result and then remained unchanged, and the protocols for lateral cyclic loading presented in [33] were adopted in this study, as depicted in Fig. 4. In the elastic range, the tests were performed under force-control, and two cycles were imposed at the load levels of $0.25Q_c$, $0.5Q_c$ and $0.7Q_c$, where Q_c is lateral capacity estimated by the formulae for capacity of CFDST members [30]. In the subsequent inelastic range, the tests were done under displacement-control, and the displacement levels of $1.0\delta_y$, $1.5\delta_y$, $2.0\delta_y$, $3.0\delta_y$, $5.0\delta_y$, $7.0\delta_y$, $8.0\delta_y$ and $11.0\delta_y$ were applied to the specimen, where δ_y ($=Q_c/K_{sec}$) is yield displacement, and K_{sec} is secant rigidity obtained by the first cycle of lateral load vs. mid-span lateral displacement hysteretic curves while $0.7Q_c$ is reached. Three cycles were imposed at the displacement levels of $1.0\delta_y$, $1.5\delta_y$ and $2.0\delta_y$, while two cycles were imposed for the remaining displacement levels.

3. Experimental Results and Discussion

3.1. Observed Damage Process and Failure Modes

It was observed that, the overall damage process of the FCFST specimens, indicating by the morphologic change of the outer tube at the control section, was analogous to that of the accompanying CFST specimen. Fig. 5 illustrates the representative damage process of specimen S30-4-0.05. Before reaching $0.7Q_c$, there was no apparent damage to the specimen, and during the $\pm 1.5\delta_y$ cycles the yielding of outer tube flanges was attained. With increasing lateral displacement to $\pm 2.0\delta_y$, slight outward local buckling (LB) appeared on the outer tube flanges; however, the LB disappeared when the lateral displacement reversed to zero. During the $\pm 3.0\delta_y$ cycles, the lateral load reached its peak, and simultaneously the LB of outer tube flanges became more severe. Upon further deforming, the lateral load began to decline and the outward LB of side walls in the outer tube

occurred during the $\pm 5.0\delta_y$ cycles. When the lateral displacement reached $\pm 7.0\delta_y$, the outward LB of flanges and side walls in the outer tube were connected to each other, resulting in the LB of corners in the outer tube. Finally, while during the $\pm 8.0\delta_y$ cycles, fracture occurred to the locally buckled corners in the outer tube.

The overall failure modes of the specimens after completing the tests are illustrated in Fig. 6. It should be noted that, the mid-span deflections of the specimens in the pictures were not pronounced, as the actuator was returned to the zero-load position to facilitate the removal of the fixture and the installation of the next specimen. For all the specimens, significant outward LB of walls and corners in the outer tube, either symmetrically or asymmetrically distributed in the vicinity of the fixture, were observed, and the LB form of the outer tube also shows some difference with the change of test variables. Similar phenomena were also observed in the previous tests of CFST and CFDST beam-columns under lateral cyclic loading [32, 34, 35]. This may be due to the drift of the actuator front end induced by the cyclic axial contraction of the specimens, as well as the random distribution of initial material and geometric imperfections of the specimens. Moreover, for the FCFSST specimens with a lower α_n and n as well as the reference CFST specimen, fracture of locally buckled corners occurs to the outer tube, considering that the plastic deformability of corners in the steel SHS is reduced after cold-forming process, resulting in the low-cycle fatigue destruction under cyclic loading.

Fig. 7 shows the typical failure modes of the sandwich concrete and inner tubes. It is shown that, in general, crushing and cracking coexist at the area corresponding to the outward LB of the outer tube, as alternating tension and compression happen to the sandwich concrete while subjected to cyclic loading, and under the combined compression and flexure, a number of longitudinal or transverse cracks also appear in the sandwich concrete away from the outward LB area of the outer tube. Meanwhile, χ and n are the key factors that determine the failure degree of the sandwich concrete, that is, with the increase of χ and n , the crushing/cracking degree of the sandwich concrete increased and accompanied by more and finer longitudinal cracks. For all the specimens, all the inner tubes produce the inward LB at the region corresponding to the outward LB of the outer

tube and crushing/cracking of the sandwich concrete, due to the restrictions of the sandwich concrete to their outward deformation, which is different from the interval outward and inward LB of the steel SHS/CHS [36, 37]. Generally, α_n has no obvious influence on failure modes of the inner tubes, while a larger χ and n leads to a severer inward LB of the inner tubes, and fracture of the inner tubes also occurs to the FCFSSST specimens with $n \geq 0.25$ and $\chi = 0.5$, due to the increased initial damage under a higher axial compression and diameter/width-to-thickness ratio. Overall, the failure modes of the inner SHSs are severer than those of the inner CHSs as the former has a worse local stability than the latter. In addition, there are no obvious interface separation between different components and/or surface scratches on the sandwich concrete and steel tubes, meaning that the components in FCFSSST specimens can generally work well together.

In summary, the failure of the outer tube, inner tubes and sandwich concrete of the FCFSSST beam-column specimens under lateral cyclic loading occur sequentially, and gradually become severer with the increase of lateral displacement. The eventual failure modes of the specimens can generally be described as a combination of the following features, i.e., outward LB of outer tube (OLB_o), fracture following LB of outer tube (FLB_o), crushing/cracking of concrete (CC), inward LB of inner tube (ILB_i), and fracture following LB of inner tube (FLB_i), as schematically illustrated in Fig. 8 and listed in Table 1. It can be found that, the failure modes of the outer tube are largely determined by n and α_n , whilst the failure modes of the sandwich concrete and inner tubes depend more on χ and n .

Avoiding or delaying local buckling of the steel tubes is the key target in the seismic design of the FCFSSST members. Hence, the relevant variables should be rationally controlled in the design, among them, the most significant one is the axial compression level, followed by the width-to-thickness ratio of outer steel tube (equivalent to α_n), and then the width (diameter)-to-thickness ratio of inner steel tubes. The aim is to ensure that the outer steel tube providing confinement to the sandwich concrete does not undergo local buckling under earthquake action, or even if the tube wall does experience local buckling, it does not extend to the tube corners. Additionally, when the variables exceed the limits, considering setting stiffeners to the outer walls of the outer tube can improve the seismic

performance of such composite members.

3.2. Load Vs. Deformation Responses

Fig. 9 exhibits the measured lateral load (Q) vs. mid-span lateral displacement (δ) hysteretic curves, where ‘OT’ and ‘IT’ are abbreviations for outer tube and inner tube, respectively. In addition, the characteristic events, corresponding to points A to G on the $Q - \delta$ hysteretic curves, during the whole destruction process under cyclic loading (typically shown in Fig. 5) are also clearly demonstrated, where events A and C are identified using the measured longitudinal strains on the outer and inner tubes, events B (including the corresponding load Q_{LB}), E, F and G are recognized based on the observations of specimen appearance, and event D of reaching lateral capacity (Q_{ce}) corresponds to the peak load captured on the $Q - \delta$ hysteretic curves. It should be noted that, due to the premature failure of the strain gauges on the inner tubes, event C of some specimens is not captured. Generally, for the FCFSST specimens, before Q_{ce} is reached, events A~C occur successively and LB of inner tube should have occurred but not visible, whilst after achieving Q_{ce} , due to the intensify of damage, events E and F appear sequentially and event G occurs to a small number of the specimens. These results show that the capacity continues to improve after the yielding and subsequent LB of the outer and inner tubes, meaning that Q_{ce} of the FCFSST specimens is controlled by the properties of the sandwich concrete. However, for specimen C33-3-0.25, event B occurs before event A, which is significantly different from other specimens, considering that the width-to-thickness ratio of its outer tube (B_0/t_0) is obviously larger than the limit of 42.7 (i.e., $52\sqrt{235/f_{y0}}$) calculated according to [38], at which no LB occurs.

Overall, the measured $Q - \delta$ hysteretic curves are stabilized and do not show the characteristics of pinching, which is similar to the results of previous tests on CFST and CFDST beam-columns [23, 24, 32, 35]. While α_n and n kept constant [Figs. 9(a)~(c) and Figs. 9(e), (i) and (k)], χ has a moderate effect on the form of the $Q - \delta$ hysteretic curves, whilst both FCFSST specimens have plumper $Q - \delta$ hysteretic curve than the reference CFST specimen, considering that the concrete with a lower strength and ductility is replaced by the steel tubes with better mechanical properties.

For the FCFSST specimens under the same χ and α_n [Figs. 9(d)~(f) and Figs. 9(e), (i) and (k)], a higher n leads to a worse plumpness of the $Q - \delta$ hysteretic curve and a rapider capacity reduction after reaching Q_{ce} . This is due to the fact that, a severer initial damage of all components and a larger second-order effect is caused by a higher axial compression, which accelerates the failure process of the composite specimens under lateral cyclic loading. Moreover, when χ and n remain unaltered [Figs. 9(b), (e) and (g)], the FCFSST specimens with a larger α_n have a plumper $Q - \delta$ hysteretic curve, as the confinement received by the sandwich concrete from the outer tube enhances as α_n increases when the inner tubes provide reliable supports for the sandwich concrete [15, 23].

The recorded $Q - \delta$ backbone curves under different variables are shown in Fig. 10. The obtained lateral capacity (Q_{ce}) of the specimens, as listed in Table 1, is calculated as the average value of two peak loads. Generally, the trend of all $Q - \delta$ backbone curves in both loading directions is identical, and with increasing δ the load (Q) successively experiences approximate linear and subsequent nonlinear increase before reaching Q_{ce} and rapid decline after reaching Q_{ce} . Fig. 10(a) shows that, χ has little effect on the first two development phases and the third development phase in one loading direction of the $Q - \delta$ backbone curves, considering that the variation in χ of the specimens also leads to the change of the cross section, that is, the decrease in the area of sandwich concrete is accompanied by the increase in the area of inner tubes, and vice versa. Generally, the reduction in specimen performance caused by the decreased contribution of one material and the improvement in specimen performance caused by the increased contribution of the other material can counterbalance each other. While in the other loading direction, the load drop rate after reaching Q_{ce} when $\chi=0.50$ is evidently quicker than that when χ of 0.22 and 0.34. This may be due to the insufficient compaction of the sandwich concrete or the subtle discrepancy in position of the two inner tubes on the first compression side of the specimen, leading to faster crushing/cracking of the sandwich concrete and LB of the inner tubes. It can be seen from Fig. 10(b) that, n has a significant impact on the trend of the $Q - \delta$ backbone curves, and as n increases, the slope of the curves (i.e., tangential stiffness) before achieving Q_{ce} slightly increases; however, the lateral displacement corresponding

to Q_{ce} obviously decreases, and the load drop rate after reaching Q_{ce} becomes greater, due to the fact that the larger axial compression makes the sandwich concrete more dense, and the second-order effect speeds up the damage process of all the components, especially after reaching Q_{ce} . Additionally, when α_n is the same and χ and d_e are close, the FCFST specimens with inner SHSs have a slightly higher $Q - \delta$ backbone curve than those with inner CHSs, but the difference in the backbone curves is very minor, which differs from the marked disparity in the backbone curves exhibited by the CFDST beam-column specimens with outer CHS and SHS [23], as the confinement of the sandwich concrete by outer CHS is significantly superior to that by outer SHS. This can be explained by the fact that, although both the area and yield strength of the inner SHSs are slightly higher than those of the inner CHSs, the increase in the area of inner SHSs leads to the reduction in the area of sandwich concrete, thereby reducing the combined contribution of inner SHSs and sandwich concrete to the performance of the specimens. Fig. 10(c) indicates that, the higher α_n is, the larger Q under the same δ is, because the composite stiffness and the confinement to the sandwich concrete from the outer tube enhance with the increase of α_n [15, 23]. Generally, within the variable range of the present tests, the influence of α_n on the load drop rate after reaching Q_{ce} is moderate. As is shown in Fig. 10(d), both FCFST specimens have a larger slope of the curve (i.e., tangential stiffness) before achieving Q_{ce} and higher Q_{ce} together with the corresponding lateral displacement than the reference CFST specimen, meaning that replacing concrete core in the CFST is an effective scheme to improve the seismic performance, although the lateral load of the former drops slightly faster than that of the latter after Q_{ce} is reached.

The $Q - \varepsilon$ (strain) hysteretic curves of a typical specimen are displayed in Fig. 11, in which ‘L’ and ‘T’ in brackets stand for longitudinal and transverse strain respectively, and ε_{yo} (ε_{yi}) is yield strain of outer (inner) tube. Both longitudinal and transverse strains of the outer and inner tubes are relatively small before tube yielding, and all the measured strains are fully developed after yielding and subsequent LB of the tubes. For the outer tube, the strains at the flanges and near the corners are greater than those at the half-height section, as the measuring points are far away from the centroidal

axis, and generally the strains at the flanges are higher than those near the corners, because the LB occurs first to the flanges (Fig. 9). For the inner tubes, the strains at the circular arc vertex are lower than those at the flanges and near the corners of the outer tube as the measuring points are closer to the centroid axis; however, the transverse strains develop more fully than the longitudinal strains, which may be due to the deviation between the location of the measuring points and the LB position of the inner tubes.

Fig. 12 exhibits the effect of test variables on the $Q - \varepsilon$ hysteretic curves in a typical position (i.e., point ob). Generally, for the FCFSSST specimens, a smaller χ and a larger α_n result in a more sufficient strain development, mainly due to a greater distance between the centroidal axis and the neutral axis of the cross-section and a better suppression of LB of the outer tube, respectively. Meanwhile, n has a moderate impact on the development of transverse strains; however, the lateral load vs. longitudinal strain hysteretic diagrams deviate to compressive side with increasing n , because a larger axial compression produces a higher initial compressive longitudinal strain. Moreover, the presence of the inner tubes does not cause significant difference in strain development between the FCFSSST specimens and the reference CFST specimen.

The evolvement of axial shortening (u_a) of the specimens under different variables is presented in Fig. 13, where δ_{ye} is the measured yield displacement, which is defined below. Overall, the axial shortening (u_a) develops symmetrically while subjected to lateral cyclic displacements (δ), although there are certain fluctuations in the evolvement of u_a due to the randomness of the failure location and sequence as well as slight deviation of the testing set-up. In addition, the variation of u_a is limited before δ_{ye} is reached, while the u_a increases rapidly after achieving δ_{ye} . It can also be observed that, for the FCFSSST specimens, χ has little influence on the evolvement of u_a , and when δ/δ_{ye} kept constant, the u_a and its gradient after reaching δ_{ye} increase as α_n decreases and n increases, which mainly causes the reduction of stiffness and the aggravation of damage, i.e., the weak ability to restrain axial shortening. In addition, due to the inner tubes' contribution to stiffness, the FCFSSST specimens lead to slower early evolvement of u_a than the reference CFST specimen;

however, in the later stage, the difference in the evolvement of u_a between two types of composite specimens is gradually eliminated due to the gradual severity of the inner tubes' damage.

3.3 Lateral Capacity and Cross-sectional Flexural Stiffness

The columns in Fig. 14 demonstrate the variation in Q_{ce} . In the same condition, the Q_{ce} of the FCF SST specimens first goes up slightly and then goes down with the improvement in χ due to the inconsistency in the properties of the inner tubes, and reduces as n increases and α_n decreases due to the increased second-order effect and the decreased confinement of the outer tube to the sandwich concrete. In addition, the FCF SST specimens having inner CHSs result in slightly higher Q_{ce} than those having inner SHSs. On average, the ratio of Q_{ce} of the FCF SST specimens with χ of 0.34 and 0.50 to that of the composite specimen with $\chi=0.22$ is 1.02 and 0.96, respectively, the Q_{ce} of the FCF SST specimens having inner CHSs (SHSs) with n of 0.25 and 0.50 is 14.4% (5.5%) and 32.9% (26.5%) lower than that of the relevant FCF SST specimen with $n=0.05$, and the FCF SST specimens with α_n of 0.085 and 0.111 result in 15.3% and 29.8% higher Q_{ce} than their accompanying specimen with $\alpha_n=0.068$, respectively. Furthermore, while α_n and n kept constant, both FCF SST specimens have a higher Q_{ce} than the reference CFST specimen as part of concrete core is replaced by steel tubes with better mechanical properties, and the improvement percentage is 20.8%. Generally, the variation of Q_{LB} across specimens is similar to that of Q_{ce} .

With respect to the cross-section with strain gauges, the moment (M) vs. curvature (ϕ) backbone diagram of the specimens can be obtained according to the recorded $Q - \delta$ backbone curve, together with the measured extreme-fibre longitudinal strains. The formulae are expressed as:

$$M = Q/2 \cdot (L_e/2 - 60) + N_0 \cdot \delta \quad (4)$$

$$\phi = (\varepsilon_t - \varepsilon_c)/B_o \quad (5)$$

where, ε_c and ε_t are extreme-fibre compressive and tensile longitudinal strain measured at points oa and oe, respectively.

As a result, the cross-sectional flexural stiffness $[(EI)_e]$, defined as the ratio of $0.2M_{ce}$ to its corresponding curvature on the $M - \phi$ backbone diagram [39], is adopted to assess the ability to

resist the bending deformation of the specimens, in which M_{ce} is flexural capacity corresponding to Q_{ce} , and the values of the obtained $(EI)_e$ of the specimens are given in Table 1 and further comparatively illustrated in Fig. 14 using diamond points. Overall, the $(EI)_e$ of the FCF SST specimens reduces with the augment of χ , but the reduction is not very high, that is, the $(EI)_e$ for $\chi=0.34$ and $\chi=0.50$ is 2.2% and 4.2% lower than that for $\chi=0.22$, respectively. It should be noted that, the $(EI)_e$ under varied χ mainly depends on the geometry of the inner tubes. The $(EI)_e$ of the FCF SST specimens enlarges as α_n and n increase, which is consistent with previous experimental findings of CFST and CFDST beam-columns under lateral cyclic loading [23, 32]. This is due to the fact that the moment of inertia of the outer tube increases with the increment of α_n , and the increment of axial compression can improve the compaction of the sandwich concrete, and thus reduce the effect of the inherent defects of concrete. Simultaneously, consistent with the pattern embodied in the Q_{ce} , the FCF SST specimens having inner CHSs possess slightly higher $(EI)_e$ than those having inner SHSs. The $(EI)_e$ of the FCF SST specimens with $\alpha_n=0.085$ and $\alpha_n=0.111$ is 8.8% and 27.5% higher than that of the relevant FCF SST specimen with $\alpha_n=0.068$, and the $(EI)_e$ of the FCF SST specimens having inner CHSs (SHSs) with n of 0.25 and 0.50 is 5.7% (2.9%) and 9.4% (6.0%) higher than that of the FCF SST specimen with $n=0.05$, respectively. It can be concluded that α_n is the variable that notably affects the cross-sectional flexural stiffness of the FCF SST beam-columns. In addition, the two FCF SST specimens respectively produce 9.5% and 5.0% higher $(EI)_e$ than the reference CFST specimen, meaning that replacing partial concrete with multiple steel tubes can also improve the cross-sectional flexural stiffness of the FCF SST beam-columns.

3.4 Stiffness Degradation Based on $Q - \delta$ Hysteretic Curves

According to the Chinese specification for seismic test [40], the stiffness degradation of the specimens under lateral cyclic loading can be evaluated by deterioration of secant stiffness from the measured $Q - \delta$ hysteretic curves. The secant stiffness during each cycle (K_i) can be expressed as follows:

$$K_i = (|-Q_i| + |+Q_i|)/(|-\delta_i| + |+\delta_i|) \quad (6)$$

where, $+Q_i$ and $-Q_i$ are positive and negative peak lateral load of i th loop, and $+\delta_i$ and $-\delta_i$ are

the corresponding lateral displacement. Only the first cycle of each displacement amplitude is considered in the calculation.

The stiffness degradation of the specimens described by the relationship between (K_i/K_1) and δ/δ_{ye} is presented in Fig. 15. As shown in the figures, when lateral displacement gradually increases, the stiffness of the specimens rapidly decreases due to the emergence and gradual accumulation of concrete damage (mainly tensile cracking and crushing), as well as the subsequent gradually aggravated buckling of the outer and inner tubes caused by the weakening of the supporting effect of the sandwich concrete on the steel tubes and the interfacial damage between different components. When δ/δ_{ye} reaches 4.0, the remaining stiffness is approximately 16.8% to 26.8% of K_1 , except for two FCF SST specimens with $n=0.5$. In the later stage, the completely damaged sandwich concrete can no longer provide support for the inner and outer tubes, and only the effective area of the steel tubes can offer stiffness for the specimens, and the remaining stiffness is only about 7% to 10% of K_1 . Meanwhile, there is no inflection point on the $K_i/K_1 - \delta/\delta_{ye}$ curves, indicating that the FCF SST beam-columns possess a stable seismic performance.

Generally, for the FCF SST specimens, χ has a moderate influence on the stiffness degradation. The $K_i/K_1 - \delta/\delta_{ye}$ curves are closely aligned under different void ratios, which is in line with the $Q - \delta$ backbone curves as depicted in Fig. 15(a). The curves in Figs. 15(b) and (c) suggest that a larger n and a lower α_n result in a faster stiffness degradation due to the increased damage of the sandwich concrete under a higher axial compression and the decreased confinement of the outer tube to the sandwich concrete, respectively. Particularly, as shown in Fig. 15(b), the FCF SST specimens with inner SHSs experience a faster stiffness degradation than those with inner CHSs. In this study, the inner SHSs have poorer local stability than the inner CHSs, which weakens the supporting effect of the inner tubes on the deformation of the sandwich concrete. It can be observed from Fig. 15(d) that before δ/δ_{ye} reaches 2.0, the stiffness degradation of the reference CFST specimen lies between that of the two FCF SST specimens. However, when $\delta/\delta_{ye} > 2.0$, the stiffness degradation of the former is slower than that of the latter, which might be attributed to the LB of the inner tubes in

the latter, leading to a weaker support for the sandwich concrete.

3.5 Ductility Coefficient and Ultimate Drift

To assess the ductility of the specimens, the ductility coefficient (μ) related to the key lateral displacements is defined as follows:

$$\mu = \delta_{ue} / \delta_{ye} \quad (7)$$

where, δ_{ue} is ultimate displacement, which equals to lateral displacement when the load in the post-peak stage drops to $0.85Q_{ce}$. In this study, the method in [41] was adopted to determine the value of yield displacement (δ_{ye}), as shown in Fig. 16, that is, the horizontal coordinate of the straight line passing through the starting point and the measured data point with vertical coordinate of $0.7Q_{ce}$ when the vertical coordinate equals to Q_{ce} .

The ultimate drift (θ_u), which was considered as an indicator that reflects the deformability of the specimens [42, 43], is also acquired in this study, and the formula is as follows:

$$\theta_u = 2\delta_{ue} / L_e \quad (8)$$

The average values of μ and θ_u for each specimen are summarized in Table 1, and the effect of variables on them are depicted in Fig. 17. It can be observed that, the results of μ and θ_u indicate similar evolutionary characteristics with the change of variables, and in this study the variable n is the most important influencing factor. The μ and θ_u of the FCFSST specimens are negatively correlated with the variables χ and n , which may be caused by the more serious damage of the sandwich concrete under the increased LB deformation of the inner tubes and second-order effect in the post-peak stage. Compared to the case of $\chi=0.22$, the $\mu(\theta_u)$ for the cases of $\chi=0.34$ and $\chi=0.50$ are reduced by 13.3%(3.7%) and 15.0%(11.3%), respectively. Similar to the CFDSTs with inner CHS in [23], the μ and θ_u of the FCFSST specimens with inner CHSs are slightly larger than those of the ones with inner SHSs. For the specimens with CHSs, the $\mu(\theta_u)$ for the cases of $n=0.25$ and $n=0.50$ are 35.6% (43.8%) and 60.9% (70.4%) lower than those for the case of $n=0.05$, respectively. In contrast, for the specimens with SHSs, the reductions are 34.4% (49.9%) and 67.7% (76.4%), respectively. Although replacing a single inner tube with multiple inner tubes mitigates the adverse

effects of the inner steel SHS, the discrepancy still persists. The μ and θ_u of the FCFSSST specimens are positively correlated with the variable α_n due to the increased confinement of the outer tube to the sandwich concrete, and the improvement in $\mu(\theta_u)$ for the cases of $\alpha_n=0.085$ and $\alpha_n=0.111$ is 11.3% (6.7%) and 15.4% (14.8%) compared to the case of $\alpha_n=0.068$, respectively. Moreover, with respect to the reference CFST specimen, both FCFSSST specimens produce lower μ and θ_u , and when the inner tubes are steel CHSs (SHSs) the reduction percentage is 22.7% (25.2%) and 8.2% (20.1%), respectively. This may be due to the fact that, in the post-peak phase, the sandwich concrete deforms inward while the serious LB happens to the inner tubes, thus accelerating destruction of the FCFSSST specimens, while the concrete core in the reference CFST specimen does not produce the inward deformation.

According to ANSI/AISC 314-16 [31], moderately ductile members are expected to have moderate plastic rotation of 0.02 rad or less, while highly ductile members are intended to have significant plastic rotation of 0.04. Although the ductility of FCFSSST specimens is somewhat lower than that of the reference CFST specimen, they still meet the requirements for highly ductile members, except for two specimens under a higher axial compression level ($n=0.5$). The θ_u of specimen C33-4-0.50 is 2.42, which meets the requirement of moderate ductility, while the θ_u of the specimen S30-4-0.50 is 1.89, which is lower than the corresponding rotation of moderate ductility. As a result, when the FCFSSST members are utilized in seismic-resistant structures, their axial compression level should be strictly controlled and not exceed 0.5, and simultaneously, it is advisable to use steel CHS for the inner tubes.

3.6. Energy Dissipation

The accumulated energy dissipation of the specimens (E_a) can be acquired by summing the area within $Q - \delta$ hysteretic diagram [40]. The specific values of accumulated energy dissipation ($E_{a,s}$), corresponding to the load dropping to $0.85Q_{ce}$, are provided in Table 1. The effect of variables on the relationship between E_a and δ/δ_{ye} of the specimens is shown in Fig. 18. The dissipated energy is negligible when δ is less than δ_{ye} , that is, the energy dissipation ability of the specimens is mainly

dependent on their post-yield performance. Generally, under the same δ/δ_{ye} , the E_a of the FCFST specimens rises as α_n increases and n decreases. This is because the improvement in the confinement of outer tube and the reduction in the second-order effect result in the simultaneous increase of lateral capacity (Fig. 14) and ductility [Figs. 17(b) and (c)]. When χ increases from 0.22 to 0.50, the E_a of the FCFST specimens demonstrates a trend of initially decreasing and then increasing, which reflects the variation of lateral capacity but does not align with the change of ductility coefficient. Simultaneously, the difference in the energy dissipation characteristics between the specimens with inner CHSs and those with inner SHSs is inconsistent when n varies. These circumstances indicate that further experimental studies on the influence rule of χ and the type of inner tube are necessary. Additionally, both FCFST specimens end up with a higher E_a than their counterpart CFST specimen, mainly due to the replacement of part of concrete core with ductile steel tubes. Specifically, for the FCFST specimens, the $E_{a,s}$ at α_n of 0.085 and 0.111 is 28.4% and 56.9% higher than that at $\alpha_n=0.068$, and the $E_{a,s}$ at n of 0.25 and 0.05 is 3.0-10.8 times (with inner CHSs) and 3.0-9.5 times (with inner SHSs) of that at $n=0.50$. Meanwhile, the $E_{a,s}$ of the FCFST specimens is 11.7% (with inner CHSs) and 21.7% (with inner SHSs) higher than that of the reference CFST specimen.

4. Prediction of Mechanical Indexes

Currently, there is no practical method for the calculation of mechanical indexes of the FCFST compression-flexure members. Therefore, the simplified methods of CFST and CFDST members in the existing design specifications [30, 38, 44] are tentatively used to try to develop the calculation method for the mechanical indexes of the FCFST beam-columns under lateral cyclic loading.

4.1. Flexural Capacity

The comparison between axial compression (N) vs. moment (M) correlation curves based on the existing design specifications and the measured capacities of the FCFST beam-column specimens in this study is demonstrated in Fig. 19, in which all three approaches in [44] that are differentiated based on B_o/t_o are included. Simultaneously, the predicted flexural capacities (M_{cc}) and their

comparison to the measured ones (M_{ce}) are presented in Table 4, where the results based on approaches (2) and (3) in [44] are presented, while the results of approach (1) without considering the contribution of concrete are ignored. In the calculation, all the material partial safety factors are set to unity. The results show that, the approaches in three design specifications all underestimate the flexural capacity of the FCFSST beam-column specimens, although EC 4 [38] and AISC [44] yield better predictions than T/CCES [30]. It is believed that, the conservative predictions are mainly due to the neglect of the strengthening effect of steel under cyclic loading [45, 46].

The previous studies on the stress vs. strain backbone curve under cyclic loading and the corresponding stress vs. strain curve under monotonic loading of different kinds of steel [45, 46] showed that, the strength of steel under cyclic loading was improved by about 20%. An experimental database, integrating extremely low cycle fatigue test data from previous researchers [45, 47-50], has been established to further validate the value of this amplification factor. The relationship between the steel yield strength under cyclic and monotonic loading from the tests is depicted in Fig. 20, where, f_y represents the steel yield strength under monotonic loading (tension), and f_y^c is the steel yield strength under cyclic loading. Overall, the ratio of f_y^c to f_y ranges from 1.0 to 1.4, with the majority of data points clustering around 1.2. Therefore, the measured results of f_{yo} and f_{yi} in Table 2 are multiplied by an amplification factor of 1.2 to take into account the influence of cyclic loading, and the recalculated $N - M$ correlation curve and capacity of the FCFSST beam-column specimens, presented with a supplementary capital letter 'M', are also presented in Fig. 19 and Table 4, respectively. It can be observed that, generally, the modified method in this study can predict the $N - M$ correlation curve and flexural capacity of FCFSST beam-columns more accurately; however, the predicted results of EC 4 and AISC are slightly higher than the measured ones, that is, the mean value and standard deviation (SD) of the ratio of $M_{cc,M}$ to M_{ce} are 1.011 and 0.031 respectively, while T/CCES provides more accurate and safe predictions, that is, the mean value and SD of the ratio of $M_{cc,M}$ to M_{ce} are 0.923 and 0.045, respectively. It is of vital importance to recognize that the extent of cyclic hardening in steel is influenced by multiple factors, such as the steel grade and strain

amplitudes. The amplification factor introduced in this research serves as an initial approximation for estimating the flexural capacity of the FCFSSST beam-columns under cyclic loading. For future specification applications, more experimental and numerical investigations should be conducted to obtain the accurate amplification factor of steel yield strength under cyclic loading.

4.2. Cross-sectional Flexural Stiffness

The cross-sectional flexural stiffness of composite sections in the aforementioned design specifications is calculated using the superposition method, while taking into account the reduction caused by concrete cracking, and the equation can be uniformly written as the following:

$$K_{ic} = E_{so} \cdot I_{so} + E_{si} \cdot I_{si} + \beta_c \cdot E_c \cdot I_c \quad (9)$$

where, E_{so} and E_{si} are elastic modulus of the outer and inner steel tubes respectively, I_{so} , I_{si} and I_c are moment of inertia of the outer steel tube, the inner steel tubes and the un-cracked sandwich concrete, respectively, and β_c is reduction coefficient. In T/CCES [30] and EC 4 [38], the value of β_c is set as 0.6 for square composite sections, while in AISC [44], the value of β_c equals to $0.45 + 3A_s/A_g$ and must be less than or equal to 0.9, in which A_s and A_g are cross-sectional area of the outer steel tube and gross area of composite section, respectively.

The calculated values of cross-sectional flexural stiffness $[(EI)_c]$ of the FCFSSST specimens are compared with the measured results $[(EI)_e]$ in Table 5. The comparison shows that, the AISC gives the best predictions, and the mean value of $(EI)_c/(EI)_e$ is 0.976 with a SD of 0.038. Overall, the results of $(EI)_c$ by the EC4 and T/CCES are somewhat conservative, and the mean value and SD of $(EI)_c/(EI)_e$ are 0.892 and 0.034, respectively. As previously mentioned, the impact of the axial compression level on the measured stiffness of the FCFSSST beam-columns is not negligible, but it is not adequately considered in the current design specifications, resulting in conservative predictions. Additionally, both EC4 and T/CCES assign a fixed value of 0.6 to the β_c for all square composite sections, disregarding the fact that different thicknesses of steel tubes lead to varying levels of confinement/support to the sandwich concrete, which in turn affects the cross-sectional flexural stiffness of the sandwich concrete, thus leading to more conservative predictions compared to AISC.

A more rational and precise method for calculating β_c , which takes into account the composite action between the steel tubes and sandwich concrete and the influence of the axial compression level, could be derived through further numerical analysis under a broader range of variables.

5. Conclusions

The seismic performance of four-opening concrete filled sandwich steel tube (FCFSST) beam-columns is studied via pseudo-static tests of typical specimens, and several important conclusions are as following:

(1) While subjected to lateral cyclic loading, the damage process of the FCFSST specimens closely resembles that of the relevant CFST specimen. The dominating failure modes of the FCFSST specimens are characterized by a combination of outward LB and fracture following LB of outer tube, crushing/cracking of sandwich concrete and inward LB and fracture following LB of inner tubes. The failure modes of the outer tube depend more on n and α_n , and those of the sandwich concrete are mainly determined by χ and n , while those of the inner tubes are mostly governed by χ and their sectional type.

(2) Generally, the load vs. deformation hysteretic curve of the FCFSST specimens is stabilized and does not exhibit the properties of pinching. For the FCFSST specimens, a lower n and a larger α_n result in a better plumpness of the $Q - \delta$ hysteretic curve, a slower stiffness degradation and a higher energy dissipation. Accordingly, for the $Q - \delta$ backbone curves, n has the greatest effect, followed by α_n , while χ and sectional type of inner tubes have moderate influence. In addition, the FCFSST specimens have a plumper $Q - \delta$ hysteretic curve, a quicker stiffness degradation when $\delta/\delta_{ye} > 2.0$ and a higher energy dissipation than the reference CFST specimen.

(3) For the FCFSST specimens, with the increase of χ , the Q_{ce} first increases mildly and then decreases, and the $(EI)_e$, μ and θ_u decrease, and the peak reduction of them is 4.0%, 4.2%, 15.0% and 11.3%, respectively. As n enlarges, the Q_{ce} , μ and θ_u reduces and the $(EI)_e$ improves, and the peak reduction of the first three indicators is 32.9%, 67.7% and 76.4%, respectively, and the peak increase of the last indicator is 9.4%. The Q_{ce} , $(EI)_e$, μ and θ_u are positively correlated with α_n ,

and the peak improvement corresponding to the maximum α_n is 29.8%, 27.5%, 15.4% and 14.8%, respectively. Moreover, the FCF SST specimens have higher Q_{ce} and $(EI)_e$, but lower μ and θ_u than the accompanying CFST specimen. It is suggested that the axial compression level of the FCF SST beam-columns in seismic-resistant structures should not exceed 0.5. Simultaneously, it is more advisable to use steel CHSs for the inner tubes from the perspective of enhancing ductility.

(4) The flexural capacity of the FCF SST beam-columns can be accurately and safely predicted by the approach in T/CCES after considering the increased steel strength under cyclic loading. Moreover, AISC provides a more accurate prediction for the cross-sectional flexural stiffness of the FCF SST specimens than the EC4 and T/CCES.

It is worthy noting that more typical tests (e.g., considering the effects of different concrete mixes) and a detailed numerical analysis under a wider range of variables shall be studied in the future to better reveal the working mechanism of FCF SST beam-columns under cyclic loading and provide a data basis for proposing a more accurate design method as well as a reasonable restoring force model. Furthermore, potential strategies to enhance the seismic performance of the FCF SST beam-columns, such as measures aimed at improving ductility and energy dissipation capacity, should be explored through experimental and/or numerical studies.

CRedit authorship contribution statement

Dong-Jie Zhang: Methodology, Data curation, Visualization, Writing - original draft, Writing - review & editing, Investigation. **You-Fu Yang:** Conceptualization, Methodology, Supervision, Project administration, Funding acquisition, Writing - review & editing. **Feng Fu:** Writing - review & editing, Methodology, Conceptualization.

Declaration of Competing Interest

The authors declared that there is no conflict of interest.

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Re:

Experimental Behaviour of four-opening concrete filled sandwich steel tube (FCFSST) beam-columns
under lateral cyclic loading

by Dong-Jie Zhang, You-Fu Yang, Feng Fu

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Declaration of interests

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

☐ The authors declare the following financial interests/personal relationships which may be considered as potential competing interests:

Figures:

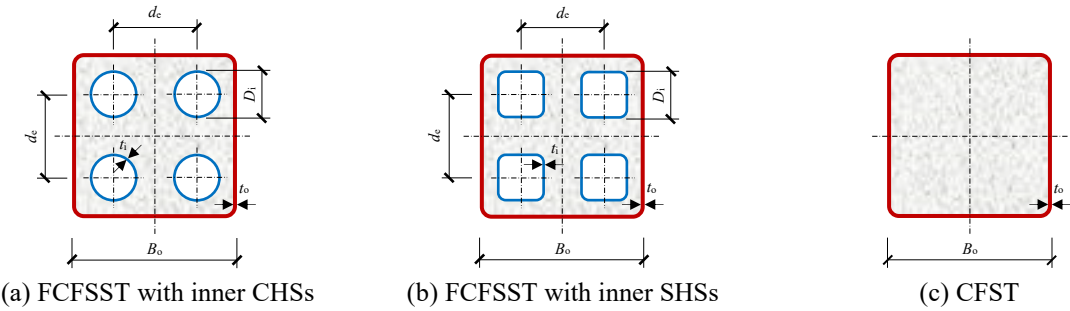


Fig. 1. Cross-section of the specimens.

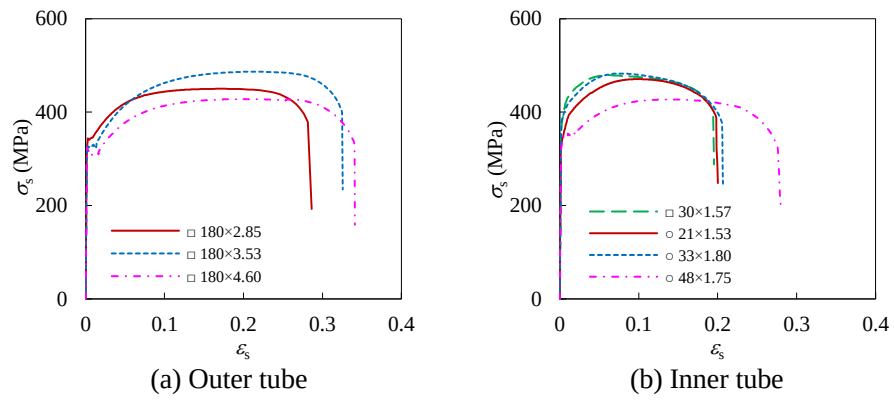


Fig. 2. The measured stress (σ_s)-strain (ϵ_s) relationship of steel tube.

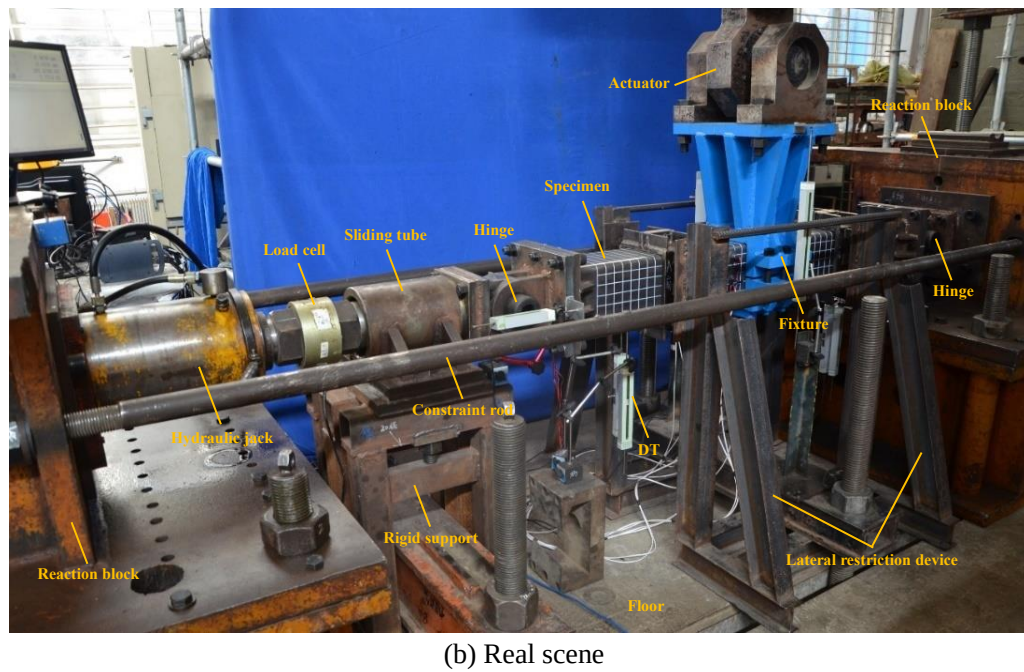
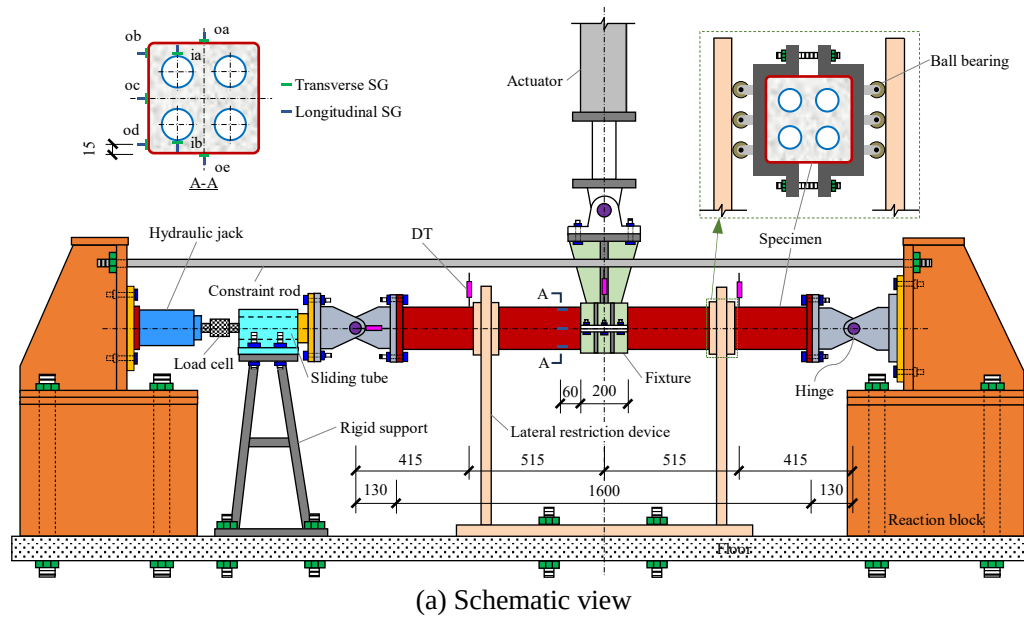


Fig. 3. Testing set-up and measurement (units: millimeters).

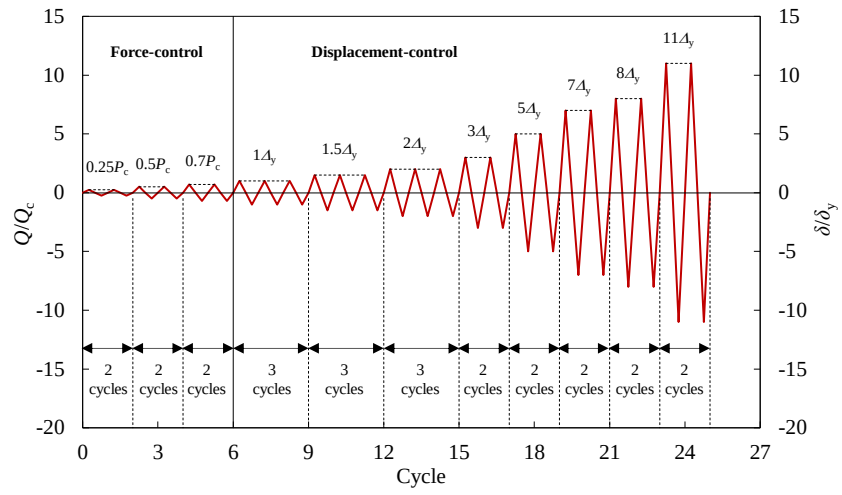


Fig. 4. Loading protocols.

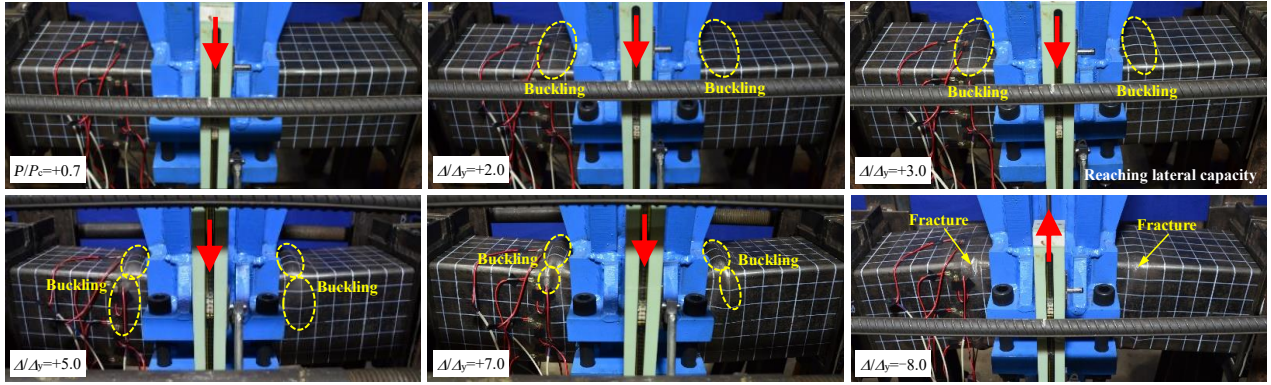
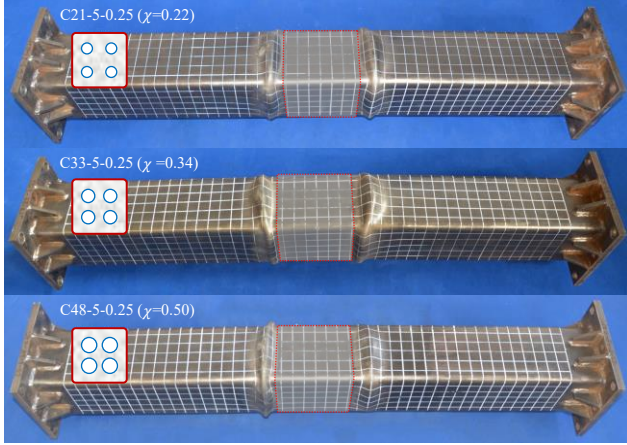
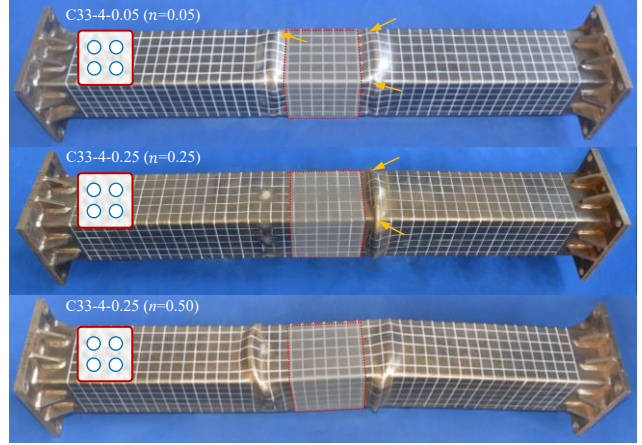


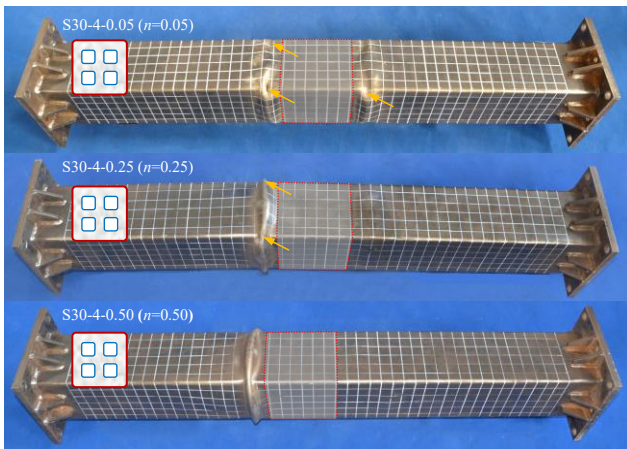
Fig. 5. Representative damage process (specimen S30-4-0.05).



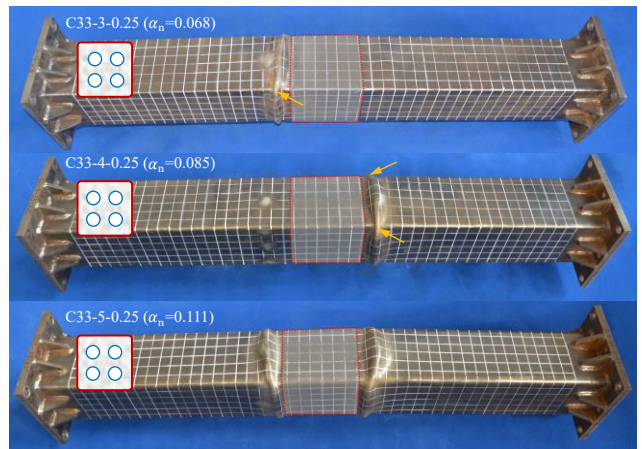
(a) Variation of χ



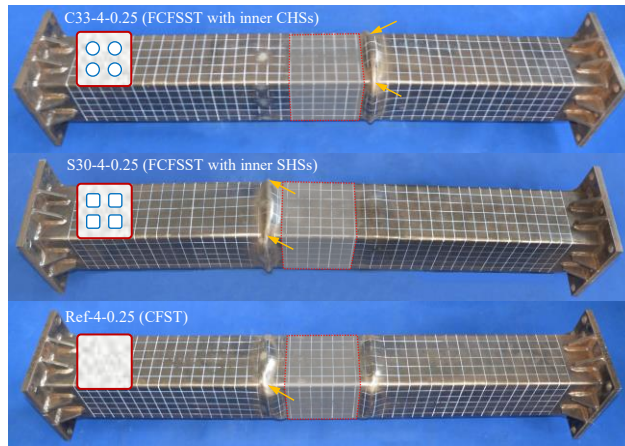
(b) Variation of n (with inner CHSs)



(c) Variation of n (with inner SHSs)

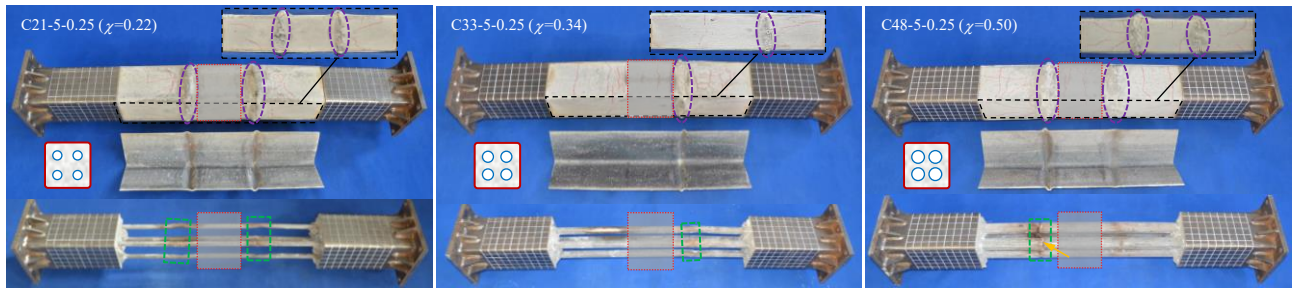


(d) Variation of α_n

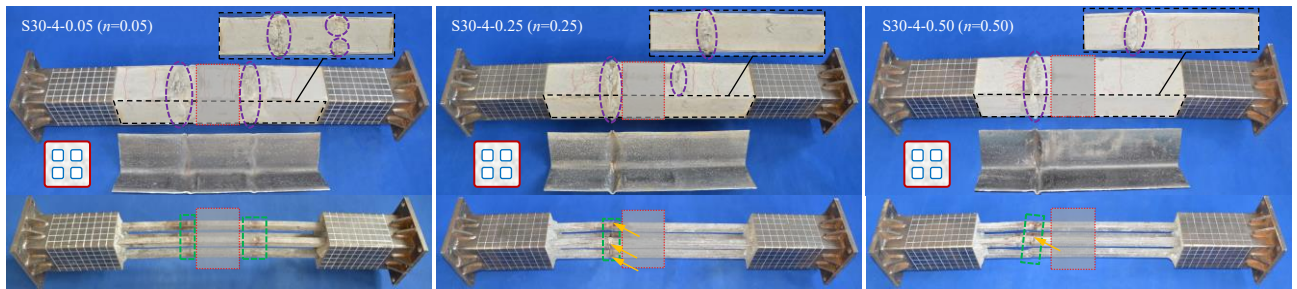


(e) Variation of cross-section

Fig. 6. Overall failure modes of the specimens. ('↘' = fracture of outer tube)

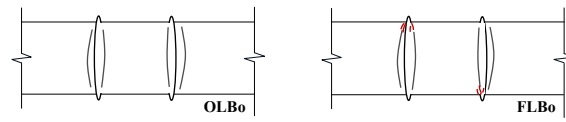


(a) Variation of χ

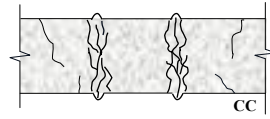


(b) Variation of n

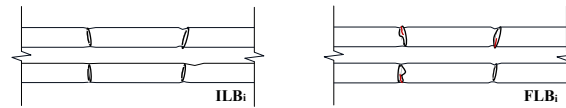
Fig. 7. Typical failure modes of the sandwich concrete and inner tubes. ('○')=crushing/cracking of concrete, '□'=local buckling of inner tube, and '↘'=fracture of inner tube)



(a) Outer tube



(b) Sandwich concrete



(c) Inner tubes

Fig. 8. Schematic diagram of typical failure modes.

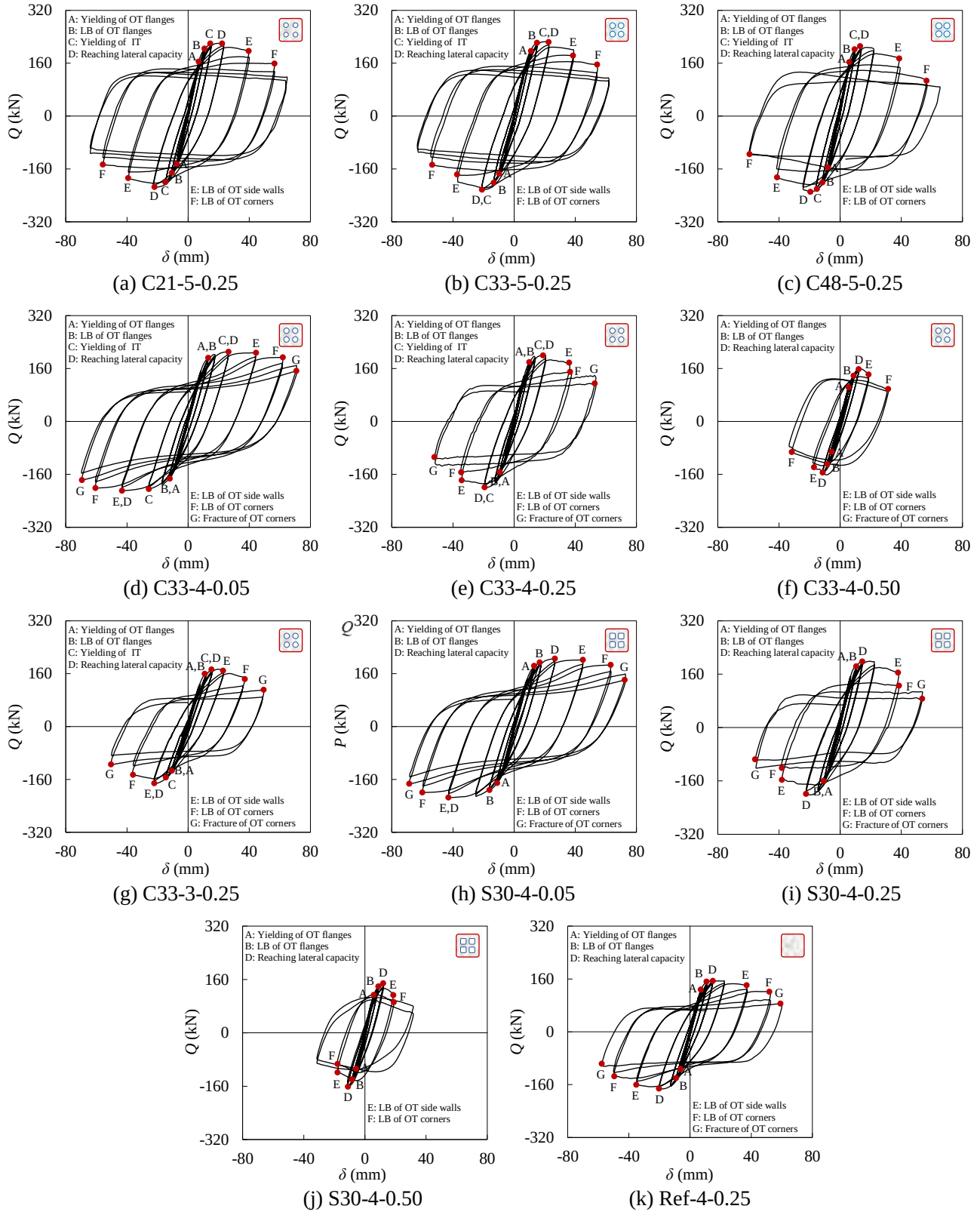


Fig. 9. Lateral load (Q) versus mid-span lateral displacement (δ) hysteretic curves.

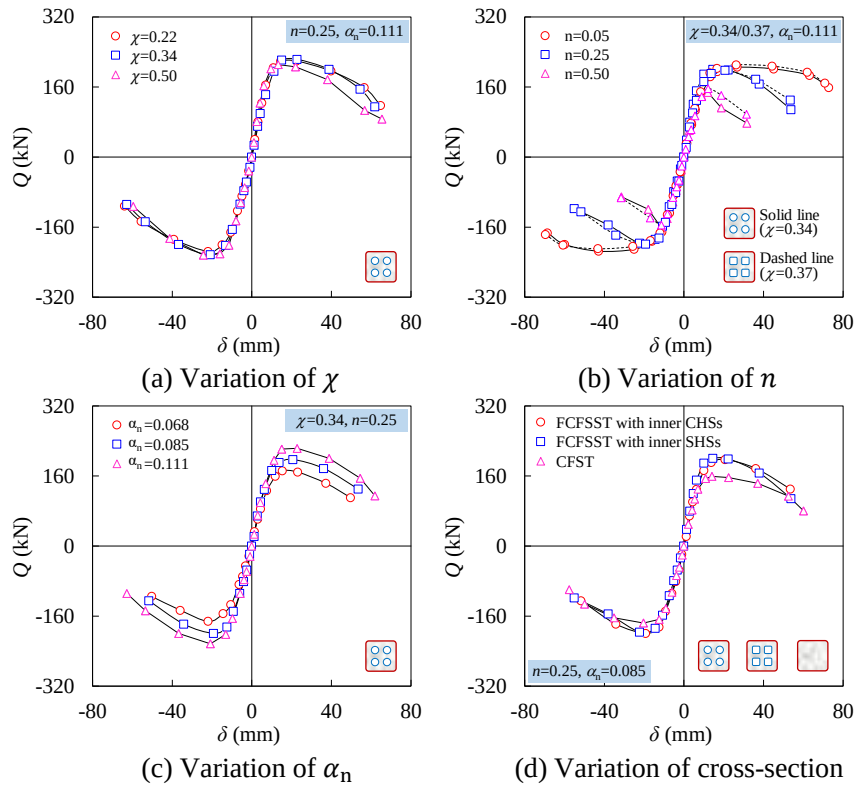
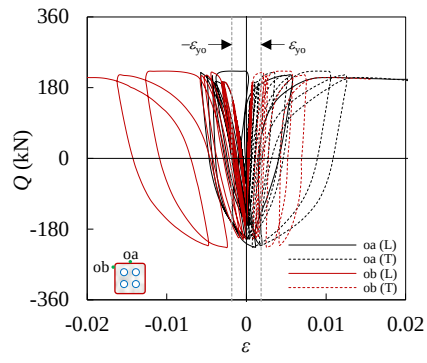
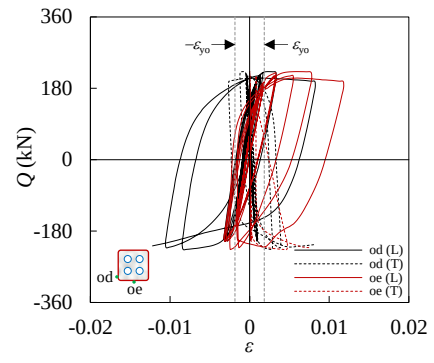


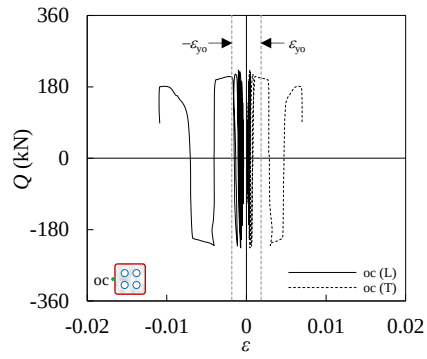
Fig. 10. Q - δ backbone curves.



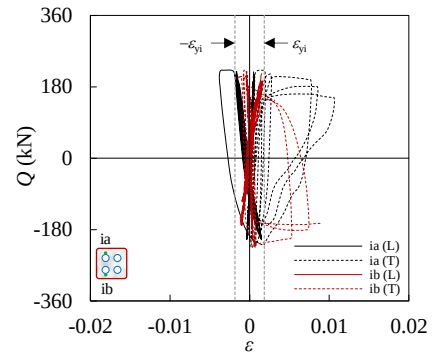
(a) Point oa and ob



(b) Point od and oe

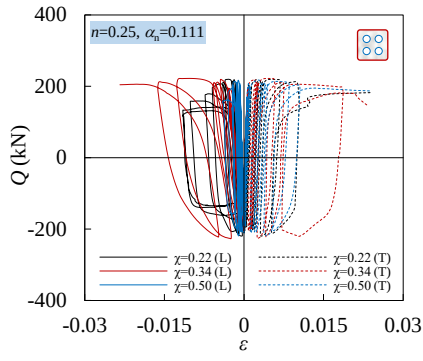


(c) Point oc

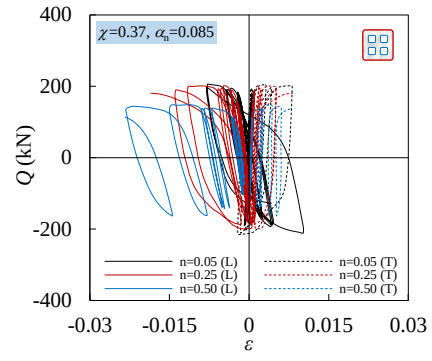


(d) Point ia and ib

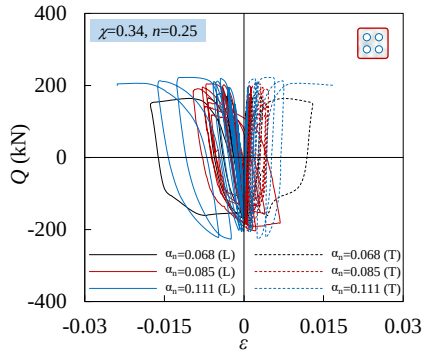
Fig. 11. Q - ϵ hysteretic curves of specimen C33-5-0.25.



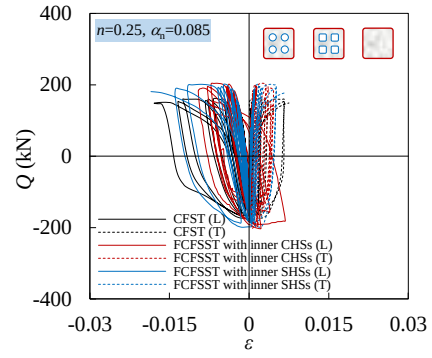
(a) Variation of χ



(b) Variation of n

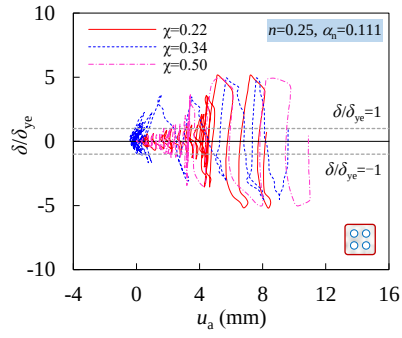


(c) Variation of α_n

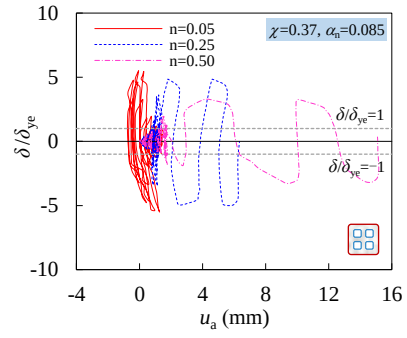


(d) Variation of cross-section

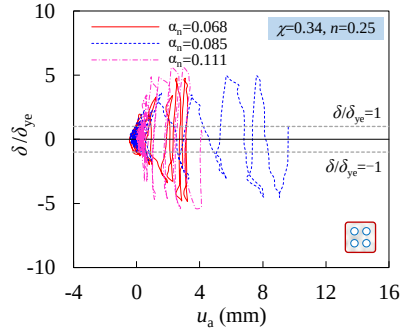
Fig. 12. Effect of variables on $Q-\varepsilon$ hysteretic curves at point ob.



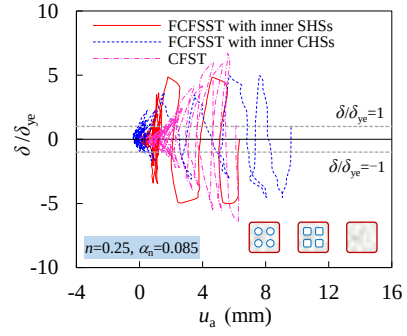
(a) Variation of χ



(b) Variation of n



(c) Variation of α_n



(d) Variation of cross-section

Fig. 13. Evolvement of axial shortenings.

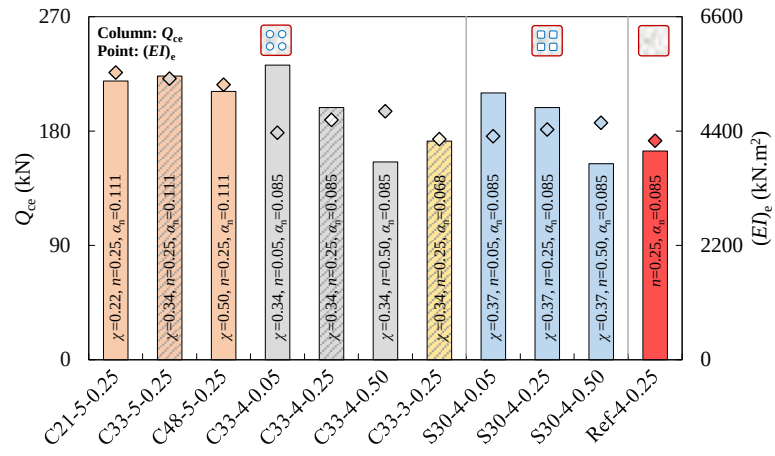
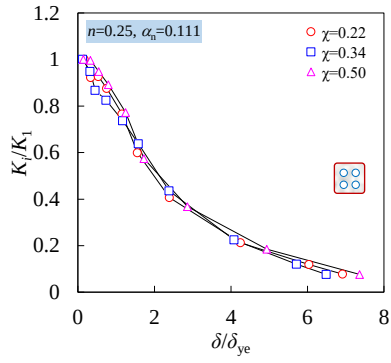
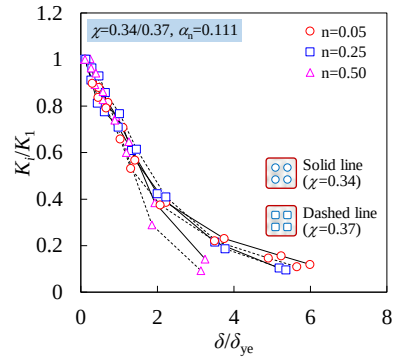


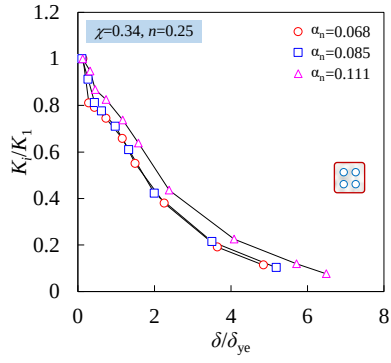
Fig. 14. Lateral capacity and cross-sectional flexural stiffness of the specimens.



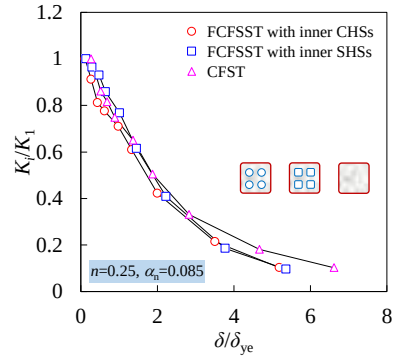
(a) Variation of χ



(b) Variation of n



(c) Variation of α_n



(d) Variation of cross-section

Fig. 15. Stiffness degradation of the specimens.

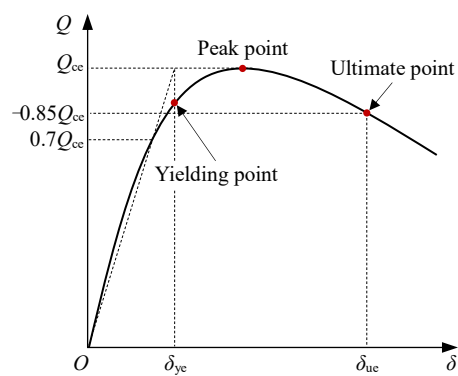


Fig. 16. Determination of key points on the measured Q - δ backbone curve.

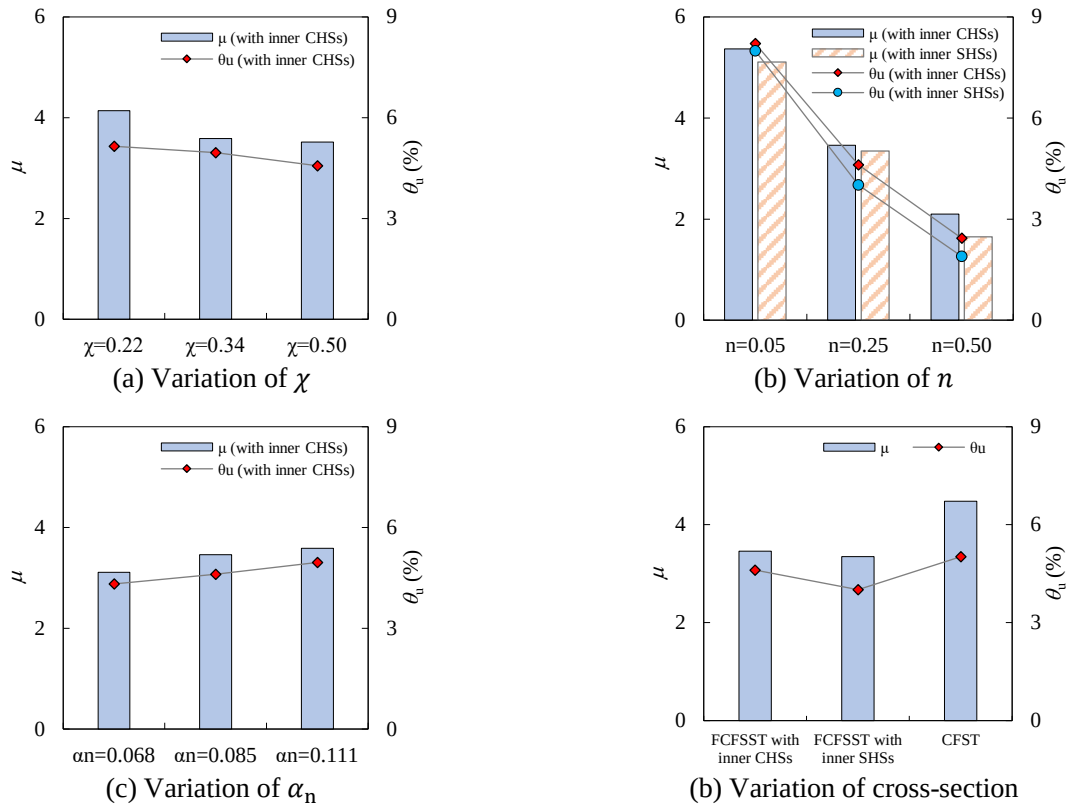


Fig. 17. Effect of variables on ductility coefficient (μ) and ultimate drift (θ_u).

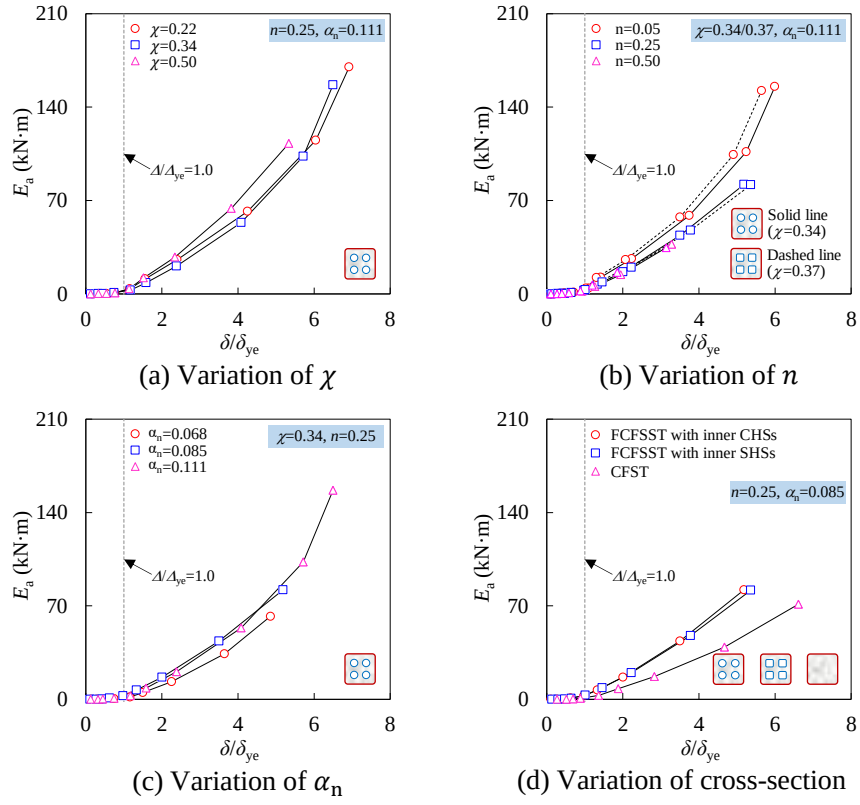


Fig. 18. Accumulated energy dissipation of the specimens.

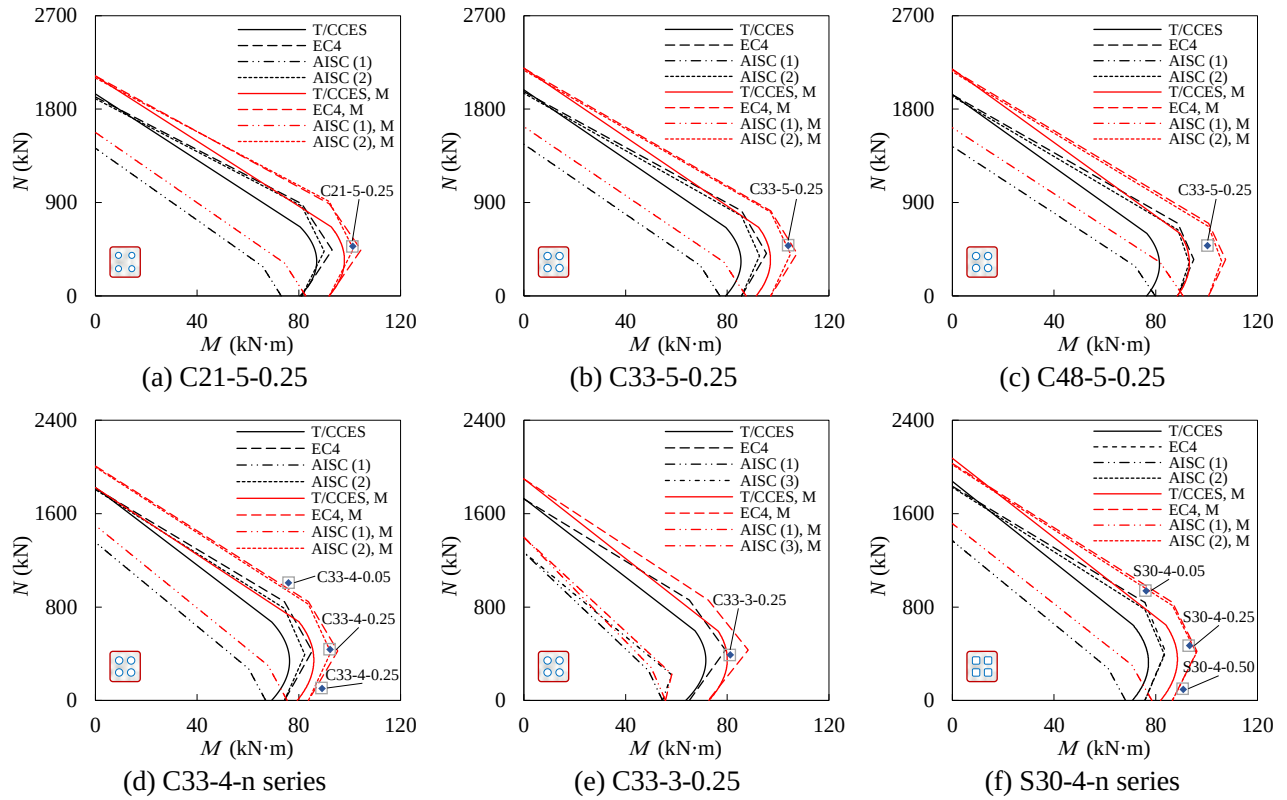


Fig. 19. Comparison between N - M correlation curves and measured capacities.

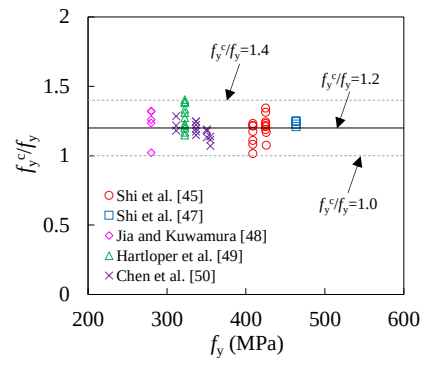


Fig. 20. Relationship between steel yield strength under cyclic and monotonic loading.

Tables:

Table 1. Details of the specimens.

No.	Label	$B_o \times t_o$ (mm×mm)	$D_i \times t_i$ (mm×mm)	d_e (mm)	f_{yo} (MPa)	f_{yi} (MPa)	χ	α_n	n	N_0 (kN)	Failure Mode ^a	Q_{LB} (kN)	Q_{ce} (kN)	$(EI)_e$ (kN·m ²)	μ	θ_u (%)	$E_{a,s}$ (kN·m)
1	C21-5-0.25	□-180×4.60	○-21×1.53	89.8	305.4	371.9	0.22	0.111	0.25	478.5	OLB _o -CC-ILB _i	188.0	219.4	5522.6	4.14	5.15	62.0
2	C33-5-0.25	□-180×4.60	○-33×1.80	77.8	305.4	368.3	0.34	0.111	0.25	484.3	OLB _o -CC-ILB _i	211.8	223.3	5403.5	3.59	4.96	53.5
3	C48-5-0.25	□-180×4.60	○-48×1.75	72.8	305.4	342.4	0.50	0.111	0.25	483.0	OLB _o -CC-FLB _i	201.5	211.2	5288.5	3.52	4.57	64.1
4	C33-4-0.05	□-180×3.53	○-33×1.80	79.9	324.6	368.3	0.34	0.085	0.05	87.3	FLB _o -CC-FLB _i	182.6	231.9	4364.9	5.37	8.21	155.5
5	C33-4-0.25	□-180×3.53	○-33×1.80	79.9	324.6	368.3	0.34	0.085	0.25	436.6	FLB _o -CC-ILB _i	166.3	198.5	4612.2	3.46	4.61	43.8
												(+20.8%) ^b	(+9.5%) ^b	(-22.7%) ^b	(-8.2%) ^b	(+11.7%) ^b	
6	C33-4-0.50	□-180×3.53	○-33×1.80	79.9	324.6	368.3	0.34	0.085	0.50	873.2	OLB _o -CC-ILB _i	133.7	155.7	4777.1	2.10	2.43	14.4
7	C33-3-0.25	□-180×2.85	○-33×1.80	81.3	348.7	368.3	0.34	0.068	0.25	390.2	FLB _o -CC-ILB _i	146.6	172.1	4239.2	3.11	4.32	34.1
8	S30-4-0.05	□-180×3.53	□-30×1.57	82.9	324.6	396.5	0.37	0.085	0.05	93.6	FLB _o -CC-ILB _i	193.0	210.0	4299.1	5.11	8.00	152.4
9	S30-4-0.25	□-180×3.53	□-30×1.57	82.9	324.6	396.5	0.37	0.085	0.25	468.0	FLB _o -CC-FLB _i	171.9	198.5	4425.8	3.35	4.01	47.7
												(+20.8%) ^b	(+5.0%) ^b	(-25.2%) ^b	(-20.1%) ^b	(+21.7%) ^b	
10	S30-4-0.50	□-180×3.53	□-30×1.57	82.9	324.6	396.5	0.37	0.085	0.50	936.1	OLB _o -CC-FLB _i	139.6	154.3	4556.6	1.65	1.89	16.1
11	Ref-4-0.25	□-180×3.53	/	/	324.6	/	/	0.085	0.25	452.1	FLB _o -CC	145.7	164.3	4213.3	4.48	5.02	39.2

Note: ^a, OLB_o=Outward local buckling of outer tube, FLB_o=Fracture following local buckling of outer tube, CC=Crushing/cracking of concrete, ILB_i=Inward local buckling of inner tube, and FLB_i=Fracture following local buckling of inner tube.
^b, The positive and negative values within parentheses represent, respectively, the percentage increase and decrease in the performance indexes of the FCF SST specimens compared to the reference CFST specimen (i.e., Ref-4-0.25).

Table 2. Properties of steel tube.

Type	$B_o \times t_o$ (mm×mm)	$D_i \times t_i$ (mm×mm)	Yield strength (MPa)	Yield strain	Tensile strength (MPa)	Elastic modulus (N/mm ²)	Poisson's ratio	Elongation after rupture (%)
Outer tube	□-180×2.85	/	348.7	0.00203	452.1	2.06×10^5	0.272	27.9
	□-180×3.53	/	324.6	0.00194	475.2	2.01×10^5	0.255	28.9
	□-180×4.60	/	305.4	0.00182	429.9	2.01×10^5	0.271	31.6
Inner tube	/	□-30×1.57	396.5	0.00247	497.9	1.93×10^5	0.292	22.4
	/	○-21×1.53	371.9	0.00218	480.2	2.05×10^5	0.286	17.4
	/	○-33×1.80	368.3	0.00215	478.5	2.06×10^5	0.309	20.3
	/	○-48×1.75	342.4	0.00198	429.9	2.08×10^5	0.276	28.9

Table 3. Mixture proportion and properties of concrete.

Mixture proportion (kg/m ³)						Properties					
Cement (P.O 42.5R)	Fly ash (Grade I)	Coarse aggregate	River sand	Tap water	Water reducer	$f_{cu,0}$ (MPa)	f_{cu} (MPa)	f_{co} (MPa)	E_c (GPa)	Slump (mm)	Spread (mm)
340	160	877	847	133	15	38.7	47.5	37.5	31.4	275	542

Table 4. Comparison between predicted flexural capacities and experimental results.

No.	Specimen ID	M_{ce} (kN·m)	T/CCES ^[30]				EC 4 ^[38] and AISC ^[44]			
			M_{cc} (kN·m)	$\frac{M_{cc}}{M_{ce}}$	$M_{cc,M}$ (kN·m)	$\frac{M_{cc,M}}{M_{ce}}$	M_{cc} (kN·m)	$\frac{M_{cc}}{M_{ce}}$	$M_{cc,M}$ (kN·m)	$\frac{M_{cc,M}}{M_{ce}}$
1	C21-5-0.25	102.0	85.8	0.841	98.2	0.963	89.9	0.882	101.4	0.994
2	C33-5-0.25	104.1	84.1	0.808	97.3	0.935	92.5	0.889	104.7	1.006
3	C48-5-0.25	100.4	79.7	0.793	93.1	0.927	92.4	0.920	105.2	1.048
4	C33-4-0.05	89.1	72.5	0.814	83.3	0.935	77.1	0.865	88.7	0.996
5	C33-4-0.25	92.2	75.7	0.821	80.9	0.878	82.4	0.894	92.5	1.004
6	C33-4-0.50	76.4	57.2	0.748	66.4	0.868	73.4	0.960	80.3	1.051
7	C33-3-0.25	81.2	71.4	0.880	80.9	0.996	76.2	0.938	85.3	1.050
8	S30-4-0.05	90.9	74.1	0.816	85.2	0.937	76.0	0.837	87.5	0.963
9	S30-4-0.25	93.4	76.9	0.824	88.0	0.943	83.5	0.893	93.7	1.003
10	S30-4-0.50	76.2	55.9	0.733	64.9	0.851	70.6	0.926	77.0	1.010
Mean		/	/	0.808	/	0.923	/	0.902	/	1.011
SD		/	/	0.042	/	0.045	/	0.043	/	0.031

Table 5. Comparison between predicted and measured cross-sectional flexural stiffness.

No.	Label	$(EI)_e$ (kN·m ²)	T/CCES ^[30] and EC 4 ^[38]		AISC ^[44]	
			$(EI)_c$ (kN·m ²)	$\frac{(EI)_c}{(EI)_e}$	$(EI)_c$ (kN·m ²)	$\frac{(EI)_c}{(EI)_e}$
1	C21-5-0.25	5522.6	4657.0	0.843	5071.0	0.918
2	C33-5-0.25	5403.5	4687.8	0.868	5195.2	0.961
3	C48-5-0.25	5288.5	4688.5	0.887	5254.3	0.994
4	C33-4-0.05	4364.9	4084.0	0.936	4457.8	1.021
5	C33-4-0.25	4612.2	4084.0	0.885	4457.8	0.967
6	C33-4-0.50	4777.1	4084.0	0.855	4457.8	0.933
7	C33-3-0.25	4239.2	3731.7	0.880	4011.3	0.946
8	S30-4-0.05	4299.1	4075.3	0.948	4450.7	1.035
9	S30-4-0.25	4425.8	4075.3	0.921	4450.7	1.006
10	S30-4-0.50	4556.6	4075.3	0.894	4450.7	0.977
Mean		/	/	0.892	/	0.976
SD		/	/	0.034	/	0.038