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# Pseudo-static tests of terminal stirrup-confined concrete-filled rectangular steel tubular columns 

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#### Abstract

This paper mainly presents a pseudo-static test program on 12 terminal stirrup-confined square concrete-filled steel tube (SCFT) columns and 14 rectangular SCFT columns under constant axial pressure. The effects of various factors on the hysteretic behavior of specimens are investigated. These factors include with or without stirrups, height of terminal stirrup region, equivalent stirrup ratio, stirrup form, loading direction, height-length ratio $(L / B)$, length-width ratio $(B / D)$, axial compression ratio $(n)$ and sliding support. The failure mode, strain ratio, hysteretic curve, skeleton curve, ultimate bearing capacity, ductility, stiffness degradation, energy dissipation, as well as the residual deformation of the specimens are analyzed. The results indicate that: (1) When $n$ is relatively larger, the bidirectional stirrups can effectively delay the local buckling of steel tube and greatly increase the ultimate bearing capacity, stiffness, equivalent damping viscosity index, residual deformation rate and ductility index, and further significantly improve the seismic behavior of the rectangular SCFT columns; (2) Axial pressure can improve the confinement effect from the steel tube to the core concrete, also bidirectional stirrups can directly confine the core concrete to decrease strain ratio of the steel tube; (3) With the same value of $n$, increasing the height of terminal stirrup region or increasing the equivalent stirrup ratio can effectively improve the seismic behavior of the rectangular SCFT columns; (4) The influence of loading direction, $L / B$ and $B / D$ on the ductility of rectangular SCFT columns are not obvious.


Keywords: Rectangular concrete-filled steel tubular column; Terminal stirrup; Pseudo-static test; Seismic behavior; Ultimate bearing capacity; Ductility index

## 1. Introduction

Concrete filled steel tubular (CFT) columns have been increasingly used in bridges and high-rise buildings due to their enhanced compressive strength and stiffness, improved ductility and higher energy absorption capacity compared to the conventional steel or concrete structures. With such benefits, the use of CFT columns is becoming more commonplace and the performance of CFT columns has caught more and more research attention [1-9]. Several studies have demonstrated that circular CFT stub columns can provide sufficient constraint from the steel tube to the core concrete [1-4]. However, the flexural rigidity and flexural capacity are comparatively low and, in particular, the configuration of joints connecting the circular CFT columns and beams is complex. In comparison, the confining effect from the steel tube to the core concrete in square or rectangular CFT columns is relatively weak, despite that the section moment of inertia (therefore bending stiffness) is improved and the joint configuration is more convenient [5-9]. However, the confining effect from the steel tube to the concrete in rectangular CFT columns is relatively weak and the load-bearing capacity and ductility under seismic action are therefore reduced. The seismic behavior of rectangular CFT members is increasingly becoming a critical problem in the engineering field [10-12].

Pseudo-static tests are usually used to study the seismic performance of CFT columns, that is, axial compression and lateral cyclic load are applied to columns simultaneously. Amit H. Varma et al. [10] conducted a pseudo-static test study on 8 square CFT columns to investigate the effects of parameters include width to thickness ratio, steel yield strength and axial compression ratio $n$ on the seismic behavior of such members. The experimental results show that there is no obvious difference of the displacement ductility index when the steel ratio of cross section is changed from $7.5 \%$ to $11.0 \%$. Also, when $n$ is 0.21 , the displacement ductility index of conventional steel specimens is not obviously different from high strength steel specimens. Similarly, Liu et al. [11] conducted a seismic behavior test study on 9 square CFT columns with the steel ratio ranged from $6.9 \%$ to $12.4 \%$ under constant axial load and lateral cyclic load. Effects of $n$, width to thickness ratio, height-length ratio and concrete strength on the ductility and energy dissipation ability were studied. Han et al. [12] focused on ultimate bearing capacity and ductility on 12 square CFT columns and 18 rectangular CFT columns with the steel ratio during $8.6 \% \sim 14.5 \%$ under a pseudo-static test study. Three key parameters including $n$, length-width ratio and core concrete strength were considered in the experimental study. The results from the above studies reflected that when $n$ is more than 0.5 , the ductility and energy dissipation of the square CFT columns and rectangular CFT columns are generally low.

Moreover, many researchers have proposed different structural measures on square CFT columns in order to increase the confinement effect from the steel tube to the core concrete and improve their seismic performance. These structural measures include steel jacket welded outside the steel tube [13], encased profile steel [14], longitudinal stiffening ribs [15] and horizontal binding bars [16] arranged inside the steel tube. Mao et al. [13] proposed 3 forms of steel jackets welded to the potential plastic hinge region to delay the local buckling of the steel tube and ensure a ductile behavior of CFT members. However, due to the limitation of welding technology, the improvement of ultimate bearing capacity and stiffness is not obvious. The structural measures outside the steel tube are mainly applied to the reinforcement of the existing CFT members. On the other hand, for the new CFT members, increasing the internal steel is usually applied to improve the steel ratio. For example, Zhu et al. [14] proposed profile steel embedded in core concrete to prevent the fracture surface of concrete under failure load. When the equivalent amount of steel is increased by $97.8 \%$, ultimate bearing capacity, displacement ductility index, energy-dissipation index are increased by $9.5 \%, 15.4 \%, 13.5 \%$. Zhang et al. [15] proposed longitudinal stiffening ribs on 2 inner faces or 4 inner faces of square steel tubes. It was found that when $n$ is 0.4 and 0.5 , compared to the specimen of 2 stiffening ribs, the specimen with 4 stiffening ribs have no obvious improvement in the ultimate bearing
capacity compared to the specimen with 2 stiffening ribs. However, its displacement ductility index is increased by $30 \%$. Wang et al. [16] proposed a measure by bolting horizontal binding bars inside the square steel tubes to postpone their local buckling and improve the seismic behavior of CFT specimens. The results show that when $n$ is 0.2 , the ultimate bearing capacity is almost unchanged while the ductility index is increased by $67 \%$. Moreover, when $n$ is 0.6 , the ultimate bearing capacity is increased by $10 \%$ and the ductility index is increased by $30 \%$.

However, it is difficult to perform the welding work for large-dimensional CFT columns due to their too thick steel tube in practical project. Consequently, the thin-walled rectangular CFT columns have been widely used. However, the latters' low steel ratio will weaken their seismic performance. In order to improve the axial compressive performance of thin-walled square CFT stub columns, Ding et al. [17] conducted a comparison study of four structural measures including studs, circular stirrups, rhombus stirrups and bidirectional stirrups, based on which they proposed a way of welding the bidirectional stirrups to the inner wall of the square steel tube. This method was proved to exert the most effective constraint on the core concrete and hence was applied to round-ended CFT stub columns under axial compression [18]. Similarly, it can also be applied to the rectangular CFT columns for the study on seismic performance.

It is known that the weak region of a CFT frame column is located at its terminal section, the idea of stirrups encryption in joint area of reinforced-concrete structure can be applied to CFT columns. In order to reduce the amount of steel, improve the economic efficiency and optimize the construction, the authors put forward the terminal stirrup-confined rectangular CFT (SCFT) columns in which the bidirectional stirrups are welded inside the rectangular steel tube at the columns ends with large bending moment. For large-dimensional CFT columns, it is convenient for operators entering the steel tube and welding stirrups only at the columns ends.

In conventional standards, storey height and storey number are limited in order to limit $n$ of the columns and ensure their seismic performance. However, in actual high-rise and super high-rise buildings, $n$ of columns is often very large, even reaching 0.8 . The aim of this study, therefore, is to focus on the advantage of rectangular SCFT columns under high $n$ even up to 0.8 . More specifically, two objectives are included in this study: (1) to investigate the seismic behavior of rectangular SCFT columns through a pseudo-static test study on 26 specimens; (2) to study the effects of 9 main factors on the hysteretic behavior of specimens include with or without stirrups, height of terminal stirrup region, equivalent stirrup ratio, loading direction, height-length ratio $(L / B)$, length-width ratio $(B / D)$, axial compression ratio ( $n$ ) and sliding support.

## 2. Experimental investigation

### 2.1 Specimens and materials

In this test program, 26 specimens were designed, including 12 square SCFT columns and 14 rectangular SCFT columns. Each specimen consisted of concrete filled steel tubular column, top plate, bottom plate and stiffening ribs. Fig. 1 and Fig. 2 exhibit a schematic view and the actual photos of the specimens, respectively. The details of the labels and parameters of specimens are listed in Table 1. In the label of specimen, the first letter "s" or " r " represents square or rectangle, the second letter "c" means column, the third letter " h " indicates that the loading mode is hysteretic. $B$ is the length (longer side) of rectangular section, $D$ is the width (shorter side) of the rectangular section, $t$ is the wall thickness of the steel tube, $L$ is the effective height of column excluding the height of stiffening ribs. $\rho_{\mathrm{s}}$ is the steel ratio of the cross section, calculated by $\rho_{\mathrm{s}}=A_{\mathrm{s}} /\left(A_{\mathrm{s}}+A_{\mathrm{c}}\right)$, where $A_{\mathrm{s}}$ and $A_{\mathrm{c}}$ are the area of steel tube and core concrete, respectively. $a_{\mathrm{s}}, b_{\mathrm{s}}$ and $d_{\mathrm{s}}$ are horizontal spacing, longitudinal spacing and diameter of stirrups, respectively. $h_{1}$ and $h_{2}$ are height of terminal stirrup region at the bottom and top of specimens. Stirrups of $h_{1}$ mainly bear the bending moment and shear force, while stirrups of $h_{2}$ are constructional reinforcement to prevent
the core concrete from premature crushing failure. $f_{\mathrm{cu}}$ is the cubic compressive strength of concrete, $f_{\mathrm{c}}$ is uniaxial compressive strength of concrete prism. According to Ding et al. [19], the conversion relationship between $f_{\mathrm{c}}$ and concrete compressive strength $f_{\mathrm{cu}}$ is $f_{\mathrm{c}}=0.4 f_{\mathrm{cu}}{ }^{7 / 6} . f_{\mathrm{s}}$ and $f_{\mathrm{sv}}$ is the yield strength of steel tube and stirrup, respectively. $\rho_{\mathrm{sa}}$ is equivalent stirrup ratio defined as $\rho_{\mathrm{sa}}=\rho_{\mathrm{sv}} \times f_{\mathrm{sv}} / f_{\mathrm{s}}$, where $\rho_{\mathrm{sv}}$ is the stirrup ratio. $n$ is axial compression ratio, calculated by $n=N / N_{\mathrm{u}}$, where $N$ is the constant axial pressure and $N_{\mathrm{u}}$ is the nominal bearing capacity. $N_{\mathrm{u}}$ is obtained from formula $N_{\mathrm{u}}=f_{\mathrm{c}} A_{\mathrm{c}}+f_{\mathrm{y}} A_{\mathrm{s}}[11,15,16] . P^{+}$and $P^{-}$are the maximum positive and negative horizontal bearing capacity. $D I$ is the displacement ductility index obtained from the average value of positive and negative displacement ductility index. $K_{1}$ is the initial stiffness obtained from the average value of positive and negative initial stiffness.

Among these specimens, sch1 and sch3 are without stirrups while the others are with bidirectional stirrups on the cross-sections of specified spacing. Besides, sch1, sch2 and sch5 were tested without sliding support because the $n$ of sch1, sch2 is 0 and sch 5 is used to compare the effect of sliding support with sch3. Moreover, the stirrup diameter of sch8 is 8 mm and sch9 is with ring stirrups. Particularly, the $n$ of rch1~4-0.8 is up to 0.8 .

(c) Sectional forms and strain rosettes arrangement

(d) Bottom plate, stiffening ribs and stirrup welding

Fig.1. schematic view of specimens

(a) Top plate and sliding support

(b) Bottom plate and stiffening ribs

(c) Stirrup welding

Fig.2. Actual diagram of specimens
Table 1 Parameters of specimens
(a) Square SCFT columns

| Specimen | $B \times D \times 1 \times L / \mathrm{mm}$ | $\rho_{\text {s }}$ | $a_{s} / \mathrm{mm} b_{s} / \mathrm{mm} d_{s} / \mathrm{mm}$ |  |  | $h_{1} h_{2}$ | $f_{\text {cu }} / \mathrm{MPa} f_{\mathrm{c}} / \mathrm{MPa} f_{\mathrm{s}} / \mathrm{MPa} f_{\mathrm{sv}} / \mathrm{MPa}$ |  |  |  | $\rho_{\mathrm{sa}}$ |  | $\frac{N / \mathrm{kN}}{0}$ | sliding <br> support <br> without | $P^{+} / \mathrm{kN} P^{-} / \mathrm{kN}$ |  | DI $K_{1} /(\mathrm{kN} / \mathrm{mm})$ |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| sch 1 | $200 \times 200 \times 3 \times 1500$ | 0.06 | - | - | - |  | 43.2 | 32.4 | 368 | 285 |  |  |  |  | 54.62 | 53.96 | 5.02 | 3.50 |
| sch2 | $200 \times 200 \times 3 \times 1500$ | 0.06 | 100 | 50 | 6 | 200200 | 43.2 | 32.4 | 368 | 285 | 0.45\% | 0 | 0 | without | 56.36 | 57.27 | 6.12 | 3.75 |
| sch3 | $200 \times 200 \times 3 \times 1500$ | 0.06 | - | - | - | 00 | 43.2 | 32.4 | 368 | 285 | 0 | 0.4 | 835.7 | with | 61.02 | 58.54 | 3.74 | 4.11 |
| sch4 | $200 \times 200 \times 3 \times 1500$ | 0.06 | 100 | 50 | 6 | 200200 | 43.2 | 32.4 | 368 | 285 | 0.45\% 0 | 0.4 | 835.7 | with | 70.15 | 68.48 | 4.38 | 4.27 |
| sch5 | $200 \times 200 \times 3 \times 1500$ | 0.06 | 100 | 50 | 6 | 0 0 | 43.2 | 32.4 | 368 | 285 | 0.45\% 0 | 0.4 | 835.7 | without | 66.28 | 66.81 | 3.35 | 3.87 |
| sch6 | $200 \times 200 \times 3 \times 900$ | 0.06 | 100 | 50 | 6 | 200200 | 43.2 | 32.4 | 368 | 285 | 0.45\% 0 | 0.4 | 835.7 | with | 95.71 | 96.88 | 4.71 | 17.63 |
| sch7 | $200 \times 200 \times 3 \times 1500$ | 0.06 | 100 | 50 | 6 | 400200 | 43.2 | 32.4 | 368 | 285 | 0.45\% 0 | 0.61 | 1253.6 | with | 52.03 | 50.34 | 2.85 | 5.08 |
| sch8 | $200 \times 200 \times 3 \times 1500$ | 0.06 | 100 | 50 | 8 | 400200 | 43.2 | 32.4 | 368 | 504 | 1.42\% 0 | 0.61 | 1253.6 | with | 83.54 | 79.39 | 3.02 | 5.76 |
| sch9 | $200 \times 200 \times 3 \times 1500$ | 0.06 | ring | 50 | 6 | 400200 | 43.2 | 32.4 | 368 | 285 | 0.73\% 0 | 0.61 | 1253.6 | with | 48.40 | 47.23 | 2.38 | 4.68 |
| sch10 | $200 \times 200 \times 3 \times 2000$ | 0.06 | 100 | 50 | 6 | 200200 | 43.2 | 32.4 | 368 | 285 | 0.45\% 0 | 0.2 | 417.9 | with | 47.58 | 45.29 | 4.62 | 1.92 |
| sch11 | $200 \times 200 \times 3 \times 2000$ | 0.06 | 100 | 50 | 6 | 200200 | 43.2 | 32.4 | 368 | 285 | 0.45\% 0 | 0.4 | 835.7 | with | 49.70 | 49.28 | 4.08 | 2.12 |
| sch12 | $200 \times 200 \times 3 \times 2000$ | 0.06 | 100 | 50 | 6 | 400200 | 43.2 | 32.4 | 368 | 285 | 0.45\% 0 | 0.61 | 1253.6 | with | 47.81 | 48.51 | 2.63 | 2.43 |

(b) Rectangular SCFT columns

| Specimen | $\begin{gathered} B \times D \times t \times L / \mathrm{mm} \\ \hline 300 \times 200 \times 3 \times 1500 \end{gathered}$ | $\begin{gathered} \rho_{\mathrm{s}} \\ \hline 0.05 \end{gathered}$ | $a_{\mathrm{s}} / \mathrm{mm} b_{\mathrm{s}} / \mathrm{mm} d_{\mathrm{s}} / \mathrm{mm}$ |  |  | $h_{1} \quad h$ | $f_{\text {cu }} / \mathrm{MPa} f_{\mathrm{c}} / \mathrm{MPa} f_{\mathrm{s}} / \mathrm{MPa} f_{\text {sv }} / \mathrm{MPa}$ |  |  |  | $\rho_{\mathrm{sa}} \quad n \quad N / \mathrm{kN}$ | sliding <br> support | $P^{+} / \mathrm{kN}$ | $P^{-} / \mathrm{kN} \quad D I$ | /kN/mm |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| rch1 |  |  | 100 | 50 | 6 | 300200 | 43.2 | 32.4 | 368 | 285 | $0.52 \% 0.2587 .7$ | with | 147.78 | 157.654 .54 | 8.34 |
| rch2 | $300 \times 200 \times 3 \times 1500$ | 0.05 | 100 | 50 | 6 | 300200 | 43.2 | 32.4 | 368 | 285 | $0.52 \% 0.41175 .5$ | with | 134.96 | 135.944 .27 | 9.85 |
| rch3 | $300 \times 200 \times 3 \times 1500$ | 0.05 | 100 | 50 | 6 | 300200 | 43.2 | 32.4 | 368 | 285 | $0.52 \% 0.72057 .1$ | with | 117.59 | 126.282 .26 | 12.41 |
| rch4 | $300 \times 200 \times 3 \times 1500$ | 0.05 | 100 | 50 | 6 | 600200 | 43.2 | 32.4 | 368 | 285 | $0.52 \% 0.72057 .1$ | with | 128.90 | 133.902 .67 | 13.52 |
| rch5 | $200 \times 300 \times 3 \times 1500$ | 0.05 | 100 | 50 | 6 | 300200 | 43.2 | 32.4 | 368 | 285 | $0.52 \% 0.2587 .7$ | with | 110.56 | 113.164 .83 | 5.52 |
| rch6 | $200 \times 300 \times 3 \times 1500$ | 0.05 | 100 | 50 | 6 | 300200 | 43.2 | 32.4 | 368 | 285 | $0.52 \% 0.72057 .1$ | with | 84.75 | 86.962 .48 | 6.77 |
| rch7 | $200 \times 300 \times 3 \times 1000$ | 0.05 | 100 | 50 | 6 | 300200 | 43.2 | 32.4 | 368 | 285 | $0.52 \% 0.72057 .1$ | with | 165.23 | 162.112 .75 | 14.28 |
| rch8 | $300 \times 200 \times 3 \times 2000$ | 0.05 | 100 | 50 | 6 | 300200 | 43.2 | 32.4 | 368 | 285 | $0.52 \% 0.2587 .7$ | with | 117.00 | 117.694 .12 | 6.27 |
| rch9 | $300 \times 200 \times 3 \times 2000$ | 0.05 | 100 | 50 | 6 | 300200 | 43.2 | 32.4 | 368 | 285 | $0.52 \% 0.41175 .5$ | with | 107.89 | 104.543 .85 | 6.82 |
| rch10 | $300 \times 200 \times 3 \times 2000$ | 0.05 | 100 | 50 | 6 | 600200 | 43.2 | 32.4 | 368 | 285 | $0.52 \% 0.61763 .2$ | with | 105.61 | 106.342 .47 | 5.71 |
| rch1-0.8 | $300 \times 200 \times 3 \times 1000$ | 0.05 | 100 | - | - | - - | 43.5 | 32.6 | 355 | - | 2329.2 | with | 268.47 | 226.812 .07 | 23.37 |
| rch2-0.8 | $300 \times 200 \times 3 \times 1000$ | 0.05 | 100 | 50 | 8 | 600200 | 43.5 | 32.6 | 355 | 444 | $1.49 \% 0.82329 .2$ | with | 328.69 | 316.573 .71 | 41.70 |
| rch3-0.8 | $300 \times 200 \times 3 \times 1000$ | 0.05 | 100 | 50 | 10 | 300200 | 43.5 | 32.6 | 355 | 532 | $2.79 \% 0.82329 .2$ | with | 338.66 | 343.173 .70 | 39.79 |
| rch4-0.8 | $300 \times 200 \times 3 \times 1000$ | 0.05 | 100 | 50 | 10 | 600200 | 43.5 | 32.6 | 355 | 532 | $2.79 \% 0.82329 .2$ | with | 377.69 | 364.964 .17 | 43.98 |

For each specimen, the steel tube was welded from two right angle tubes which were firstly bent using the Q235 hot-rolled steel plates. The welding was performed according to the standard GB 50017-2003 [20] and the ends of the steel grooves (as the sites of welding) were kept smooth after welding. Both ends of
stirrups were firstly bent to right angle with bent length of 20 mm and then welded to the two ends of the steel tubes in a certain range. Moreover, spot welds were adopted at the intersections of bidirectional stirrups and thus they form a steel mesh.

The bottom plate and stiffening ribs were welded to the bottom of the steel tube. Then the concrete was pumped into the tube from the top and was vibrated to be well compacted. The commercial concrete of grade C40 was adopted for all the specimens. Moreover, 9 standard concrete cubes with a dimension of $150 \mathrm{~mm} \times 150 \mathrm{~mm} \times 150 \mathrm{~mm}$ were prepared and cured at the same condition as those of SCFT specimens. After 28 days of curing, the concrete had hardened completely and achieved its compressive strength. Then the cover plate was welded to the top of the steel tube. For the convenience of observation and record of failure mode, red paint was sprayed on the external surface of the steel tube and $50 \mathrm{~mm} \times 50 \mathrm{~mm}$ white grids were plotted on the surface.

Before the column tests, the cubic compressive strength $f_{\text {cu }}$ of concrete were obtained from the testing of the concrete cubes according to GB/T 50081-2002 [21]. The material properties of 3 mm thick steel plates and stirrups of diameter 6 mm and 8 mm were obtained from the tensile coupon tests according to GB/T 228-2002 [22]. The measured material properties are presented in Table 1.

### 2.2 Experimental setup and instrumentation

The pseudo-static tests on rectangular SCFT column specimens were conducted using a MTS pseudo-static test system in the National Engineering Laboratory for Construction Technology of High Speed Railway at Central South University. Fig. 3 and Fig. 4 present the schematic diagram and the actual photos of the tests. A top plate was fixed with the sliding support by bolts and to transmit the vertical axial pressure. Besides, a bottom plate was fixed with a custom-made reinforced concrete base by bolts. Then the base was fixed with rigid floor through anchor bolts. The base was strictly reinforced and debugged to eliminate any possible failure during testing. The vertical load was exerted to the specimens through a 2500 kN hydraulic jack tensioned by rebars. The oil pump was manually controlling to ensure the vertical load be stable. The horizontal low cyclic load was exerted by the MTS system hydraulic actuator through the loading chunk.


Fig.3. Schematic diagram of test

(d) Front view of rch7
(e) Side view of rch1-0.8
(f) Front view of rch1-0.8

Fig.4. Actual diagram of test
At the beginning of each test, the specimen was prepressed to a vertical load to $50 \%$ of the specified axial pressure $N$, then unloaded to 0 , after that the specimen was loaded to $N$ before the lateral force was applied. The axial pressure $N$ was kept constant during the whole test. According to JG101-1996 [23], the displacement controlled method was use for horizontal cyclic loading shown in Fig.5. One loading cycle was applied for each of the peak displacement, $(1,1.5,2,3,4,5 \sim 16) \Delta_{\mathrm{y}}$. Here $\Delta_{\mathrm{y}}$ is experimental yield displacement. Such a loading procedure was attempted until the horizontal load of the specimen decreased to $85 \%$ of the horizontal bearing capacity.


Fig.5. Displacement loading system

### 2.3 Measuring point arrangement

The horizontal load, horizontal displacement and strain of steel tube were measured during the tests. The horizontal load was collected by the MTS actuator and recorded manually by the tester. Three displacement transducers with high precision were installed at three deferent places, namely at the same height as the horizontal loading point, half of the column height and the bottom of the specimens, respectively, to measure horizontal displacement. Moreover, four strain rosettes (S1 to S 4 ) were placed on two adjacent surfaces at the bottom of the steel tube, as illustrated in Fig. 1 (c). The displacement and strain were acquired by a DH3818 static data measurement system. The local buckling deformation of steel tube, failure mode and failure location during the tests were also observed and recorded.

## 3. Experimental results and discussion

### 3.1 Damage mechanism

The damage process of the specimens was basically the same, which could be divided into 3 stages including elastic stage, elastic-plastic stage and failure stage. At the elastic stage of loading, the load-displacement curve of each specimen was basically linear. There was no obvious local buckling on the surface of the steel tube, and the strain was small. As the horizontal displacement increased, the specimens turned into elastic-plastic stage. The stiffness of them degraded and the load increased slowly. The steel tube displayed apparent local buckling above the stiffening ribs. As the test progressed, the range and degree of the local buckling was increasing continuously. When the horizontal load dropped below $85 \%$ of the ultimate bearing capacity, the specimens were failed. At this stage, the strain of steel tube increased rapidly with severe buckling (together with tearing at the corner of sections in some cases) in the region of stirrups and extended region of 50 mm above the stirrups. Moreover, the internal stirrups were snapped with crisp sound which indicated that the strain of stirrups reached the ultimate strain and their tensile strength was fully utilized. The final failure modes of the typical specimens are shown in Fig. 6.

Especially when $n$ was up to 0.8 , rch1- 0.8 (without stirrups) showed obvious axial compression, the steel tube was torn with a large area along the weld, the stirrups were snapped and the core concrete was completely crushed, which is characterized by brittle failure. Unlike this, the steel tube of rch2-0.8 ( $\rho_{\mathrm{sa}}=1.49 \%$ ) displayed apparent local buckling at the bottom. Even better, only slight local buckling occurred at the bottom of rch3-0.8 ( $\rho_{\mathrm{sa}}=1.49 \%$ ) and rch4-0.8 $\left(\rho_{\mathrm{sa}}=1.49 \%\right)$. Besides, the core concrete of rch2-0.8, rch3-0.8 and rch4-0.8 was only partially crushed, which is characterized by ductile failure. These indicate that the stirrups can effectively pull the steel pipe and confine the core concrete.

(a) sch 1

(e) sch6

(b) $\operatorname{sch} 2$

(f) $\operatorname{sch} 8$

(c) $\operatorname{sch} 3$

(g) $\operatorname{sch} 12$

(d) $\operatorname{sch} 4$

(h) rchl

(i) rch3

(j) rch4

(k) rch6
(l) rch10

(m) Steel tube of rch1~4-0.8


(n) Core concrete of rch1~4-0.8

Fig.6. Typical failure modes of specimens

### 3.2 Load-strain ratio ( $\boldsymbol{P}$ - $\boldsymbol{v}_{\mathrm{sc}}$ ) curves

Fig. 7 presents the load-strain ratio ( $P-v_{\mathrm{sc}}$ ) curves of 5 typical specimens including sch1, sch3, sch4, rch1-0.8 and rch2-0.8. As shown in Eq. (1), the $v_{\mathrm{sc}}$ is defined as the absolute value of ratio of circumferential strain to axial strain of the steel tube, reflecting the hoop constraint of the steel tube exerting on the core concrete [17, 24]. The larger the $v_{\mathrm{sc}}$ is, the stronger the hoop constraint is.

$$
\begin{equation*}
v_{s c}=\frac{\varepsilon_{p}}{\varepsilon_{a}} \tag{1}
\end{equation*}
$$

As the horizontal cyclic displacement is applied, the axial strain and circumferential strain of the steel tube varies continuously, which make the strain ratio $v_{\mathrm{sc}}$ oscillates. Besides, the maximum $v_{\mathrm{sc}}$ of the 5 specimens exceeded 0.5 in the tests. The results suggest that the steel tube of all the 5 specimens exerted the hoop constraint on the core concrete. The maximum $v_{\text {sc }}$ of 3.0 for sch3 (with $n=0.4$ ) is greater than that of 2.0 for the sch1 (without axial pressure). It is demonstrated that the axial pressure can increase the confining effect of the steel tube exerting on the core concrete. The maximum $v_{\mathrm{sc}}$ of 1.5 for sch4 (with stirrups) is less than that of 3.0 for sch3 specimen (without stirrups). Similarly, the maximum $v_{s c}$ of 3.5 for rch2-0.8 (with stirrups) is less than that of 4.5 for rch1-0.8 (without stirrups). This indicates that the maximum $v_{\text {sc }}$ of the steel tube is reduced due to the direct confining effect of the stirrups exerting on the core concrete.

(a) $\operatorname{sch} 1$ ( $n=0$, without stirrups)

(b) $\operatorname{sch} 3$ ( $n=0.4$, without stirrups)

(c) $\operatorname{sch} 4$ ( $n=0.4$, with stirrups)


Fig.7. $\boldsymbol{P}$ - $\boldsymbol{v}_{\text {sc }}$ curve of 5 typical specimens

## 4. Effects of parameters on seismic behavior

### 4.1 Seismic behavior indexes

In this paper, 5 seismic behavior indexes are analyzed including horizontal bearing capacity $P$, ductility, stiffness degradation, energy-dissipation capacity and residual deformation. The horizontal bearing capacity is obtained directly from experimental results. The ductility of the specimens is expressed by the displacement ductility index $D I$, which is defined as the ratio of the failure displacement $\Delta_{u}$ over the virtual yield displacement $\Delta_{y v}$ as shown in Eq. (2):

$$
\begin{equation*}
D I=\frac{\Delta_{u}}{\Delta_{y v}} \tag{2}
\end{equation*}
$$

The displacement ductility index is determined by the "General yield bending moment method" (also known as "tangent method") [25], as shown in Fig. 8. OC is the tangent of the $P-\Delta$ skeleton curve (ODFAB) at origin point $\mathrm{O}, P_{y v}$ and $\Delta_{y v}$ are the virtual yield load and the corresponding displacement, $P_{\max }$ and $\Delta_{\max }$ are the ultimate load and the corresponding displacement, $P_{u}$ is the horizontal failure load equal to $85 \%$ of the ultimate load in the descending range of the skeleton curve, $\Delta_{u}$ is the corresponding displacement. According to Eq. (2) and Fig. 8, the displacement ductility indices $D I$ of all specimens are listed in Table 1.


Fig.8. Ductility index obtained from general yield bending moment method
The annular stiffness $K[13,15]$ is used to evaluate the stiffness degradation of the specimens, which is obtained from Eq. (3):

$$
\begin{equation*}
K=\frac{\sum_{i=1}^{n} P^{i}}{\sum_{i=1}^{n} \Delta^{i}} \tag{3}
\end{equation*}
$$

where $P^{i}$ and $\Delta^{i}$ are respectively the peak horizontal load and the corresponding displacement of the $i$-th cycle, $n$ is the total number of hysteresis loops.

The equivalent viscous damping index $h_{e}[15,26]$ is used to estimate the energy-dissipation capacity
of the specimens, defined as Eq. (4):

$$
\begin{equation*}
h_{e}=\frac{1}{2 \pi} \frac{S_{A B C D A}}{S_{(O B E+O D F)}} \tag{4}
\end{equation*}
$$

where $S_{A B C D A}$ is the area of each hysteresis loop $A B C D$ (the purple area), $S_{(O B E+O D F)}$ are the total area of triangle $O B E$ and triangle $O D F$ (the shadow area), indicated in Fig.9.


Fig.9. Calculation of equivalent viscous damping index Residual deformation rate $r$ [27] is defined by Eq. (5):

$$
\begin{equation*}
r=\frac{O C^{i}}{O E^{i}} \tag{5}
\end{equation*}
$$

where $O E^{i}$ and $O C^{i}$ are the maximum displacement and the corresponding residual displacement of the $i$-th cycle, also indicated in Fig.9.

### 4.2. Effect of stirrups

Fig. 10 compares the effect of stirrups on the hysteresis behavior when $n$ is $0,0.4,0.8$ and the $\rho_{\text {sa }}$ is $0.45 \%, 0.45 \%, 1.49 \%$. Table 2 lists the improvement effect of stirrups on the 5 seismic behavior indexes of the specimens. When $n$ is 0 , compared to sch1 without stirrups, the hysteresis curve of sch2 is not obviously different and its $P, K_{1}$, maximum $h_{e}$, maximum $r$ are only slightly improved. However, the skeleton curve of sch 2 declines more slowly and the $D I$ increased by $21.9 \%$. When $n$ is 0.4 , compared to sch3 without stirrups, the hysteresis curve of sch4 is plumper and the skeleton curve declines more slowly. In addition, the $P, K_{1}$, maximum $h_{e}$, maximum $r$ are improved significantly. When $n$ is 0.8 , compared to rch1-0.8 without stirrups, all the 5 seismic behavior indexes of rch2-0.8 are improved significantly. It shows more obvious effect of stirrups on improving the hysteresis behavior of rectangular CFT columns under high axial compression ratio.

| Index | $n=0$ | $n=0.4$ | $n=0.8$ |
| :---: | :---: | :---: | :---: |
| Hysteretic curve |  |  |  |
|  | $\begin{array}{ccccccccc} -120 & -90 & -60 & -30 & 0 & 30 & 60 & 90 & 120 \\ \Delta / \mathrm{mm} \end{array}$ | $\begin{array}{ccccccccc} -120 & -90 & -60 & -30 & 0 & 30 & 60 & 90 & 120 \\ \Delta / \mathrm{mm} \end{array}$ | $$ |



Fig.10. Influence of stirrups on hysteresis behavior when $\boldsymbol{n}$ is $\mathbf{0 , 0 . 4} \mathbf{~ a n d ~} 0.8$
Table 2 Improved effects of stirrups on seismic behavior indexes when $\boldsymbol{n}$ is $\mathbf{0 , 0 . 4}$ and $\mathbf{0 . 8}$

| Indexes | $n=0$ |  |  |  | $n=0.4$ |  |  |  |  |  |  |  | $n=0.8$ |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | sch1 | sch2 | Improve percentage | sch3 | sch4 | Improve percentage | rch1-0.8 | rch2-0.8 | Improve percentage |  |  |  |  |
| max $\left(P^{+}, P^{-}\right)$ | 54.62 | 57.27 | $4.9 \%$ | 61.02 | 70.15 | $15.0 \%$ | 268.47 | 328.69 | $22.4 \%$ |  |  |  |  |
| $D I$ | 5.02 | 6.12 | $21.9 \%$ | 3.74 | 4.38 | $17.1 \%$ | 2.07 | 3.71 | $79.2 \%$ |  |  |  |  |
| $K_{1}$ | 3.5 | 3.75 | $7.1 \%$ | 4.11 | 4.27 | $3.9 \%$ | 23.37 | 41.70 | $78.4 \%$ |  |  |  |  |
| $\max \left(h_{\mathrm{e}}\right)$ | 0.29 | 0.31 | $6.9 \%$ | 0.23 | 0.36 | $56.5 \%$ | 0.22 | 0.36 | $63.6 \%$ |  |  |  |  |
| $\max (r)$ | 0.65 | 0.71 | $9.2 \%$ | 0.48 | 0.73 | $52.1 \%$ | 0.56 | 0.73 | $30.4 \%$ |  |  |  |  |

### 4.3. Effect of height of terminal stirrup region

Fig.s 11 and 12 compare the effect of height of terminal stirrup region $h_{1}$ on the hysteresis behavior when $n$ is $0.7,0.8$ and the $\rho_{\mathrm{sa}}$ is $0.52 \%, 2.79 \%$. It is reflected from Fig.s 11,12 and Table 1 that the hysteresis loop is plumper and the skeleton curve tends to decline more slowly when the $h_{1}$ is increased from $B(300 \mathrm{~mm})$ to $2 B(600 \mathrm{~mm})$. Compared to specimen rch3, the $P, D I, K_{1}$ of rch4 are increased from $126.28 \mathrm{kN}, 2.26,12.5 \mathrm{kN} / \mathrm{mm}$ to $133.90 \mathrm{kN}, 2.67,13.5 \mathrm{kN} / \mathrm{mm}$. On the whole, the 3 indexes are improved
by $6.0 \%, 18.1 \%, 8.0 \%$ respectively. Besides, stiffness degrades more gently. Furthermore, $h_{e}$ and $r$ are generally increased at the same loading displacement, which indicates that increasing $h_{1}$ can effectively improve the seismic behavior of specimens. Similarly, compared to specimen rch3-0.8, the $P, D I, K_{1}$ of rch4-0.8 are improved by $10.1 \%, 12.4 \%, 10.5 \%$, respectively.


(c) Stiffness degradation

Fig.11. Influence of height of terminal stirrup region on hysteresis behavior when $\boldsymbol{n}$ is $\mathbf{0 . 7}$


(e) Residual deformation

Fig.12. Influence of height of terminal stirrup region on hysteresis behavior when $\boldsymbol{n}$ is $\mathbf{0 . 8}$

### 4.4. Effect of equivalent stirrup ratio

Fig. 13 compares the effect of equivalent stirrup ratio $\rho_{\mathrm{sa}}$ on the hysteresis behavior when $n$ is 0.6 and the stirrups range is $400 \mathrm{~mm}(2 B)$. The diameter $d_{s}$ of the stirrups are 6 mm and 8 mm , and the yield strength $f_{s}$ of the stirrups are 285 MPa and 504 MPa , respectively. Thus the $\rho_{\text {sa }}$ increases from $0.45 \%$ to $1.42 \%$ and
increased by $215.6 \%$. It can be seen from Fig. 13 and Table 1 that the hysteresis loop is plumper and the skeleton curve declines more slowly when the $\rho_{\mathrm{sa}}$ is increased. Compared to sch7, the $P, D I, K_{1}$ of sch8 are increased from $52.03 \mathrm{kN}, 2.85,5.12 \mathrm{kN} / \mathrm{mm}$ to $83.54 \mathrm{kN}, 3.02,5.87 \mathrm{kN} / \mathrm{mm}$. On the whole, the 3 indexes are improved by $60.6 \%, 10.2 \%, 8.0 \%$ respectively. In addition, stiffness degrades more gently. At the early stage of loading, the difference of $h_{e}$ between sch7 and sch8 is not obvious. But at the late stage of loading, the $h_{e}$ of sch8 was significantly less than $h_{e}$ of sch7. This is because the horizontal load of sch8 declines slower than that of sch7, and the $S_{(O B E+O D F)}$ in the formula (4) is still larger, resulting in a smaller $h_{e}$. However, there is little difference between the $r$ of the 2 specimens. The above analysis contributes that increasing $\rho_{\mathrm{sa}}$ can effectively improve the seismic behavior of the specimen.

Similarly, Fig. 14 compares the effect of $\rho_{\mathrm{sa}}$ on the hysteresis behavior when $n$ is 0.8 and the stirrups range is $600 \mathrm{~mm}(2 B)$. The diameter $d_{s}$ of the stirrups are 8 mm and 10 mm , and the yield strength $f_{s}$ of the stirrups are 444 MPa and 532 MPa , respectively. Thus the $\rho_{\mathrm{sa}}$ increases from $1.49 \%$ to $2.79 \%$ and increased by $87.2 \%$. Compared to specimen rch2-0.8, the $P, D I, K_{1}$ of rch4-0.8 are improved by $14.9 \%, 12.4 \%, 5.5 \%$, respectively.


Fig.13. Influence of equivalent stirrup ratio on hysteresis behavior when $\boldsymbol{n}$ is $\mathbf{0 . 6}$



Fig.14. Influence of equivalent stirrup ratio on hysteresis behavior when $\boldsymbol{n}$ is $\mathbf{0 . 8}$

### 4.5. Effect of stirrup forms

Fig. 15 compares the effect of stirrup forms on the hysteresis behavior when $n$ is 0.6 and the stirrups range is $400 \mathrm{~mm}(2 B)$. The $\rho_{\mathrm{sa}}$ of sch 9 (ring stirrups) and sch7 (bidirectional stirrups) are $0.73 \%$ and $0.45 \%$ respectively, reduced by $38.4 \%$. However, compared to sch 9 , the hysteresis loop of sch 7 is plumper and its skeleton curve tends to decline more slowly. In addition, The $P, D I, K_{1}$ of sch 7 are increased from 48.40 kN , $2.38,4.8 \mathrm{kN} / \mathrm{mm}$ to $52.03 \mathrm{kN}, 2.85,5.1 \mathrm{kN} / \mathrm{mm}$. On the whole, the 3 indexes are improved by $7.5 \%, 19.7 \%$, $6.3 \%$, respectively. Furthermore, $h_{e}$ and $r$ of sch7 are greater than those of sch9, which state that the seismic behavior of specimen with bidirectional stirrups is superior to specimen with ring stirrups.


(d) Energy dissipation

(e) Residual deformation

Fig.15. Influence of stirrup form on hysteresis behavior

### 4.6. Effect of loading direction

Fig. 16 and Fig. 17 show the difference of loading direction on the hysteretic behavior of the specimens, when $n$ are 0.2 and 0.7 , respectively. It is explained from Fig. 16, Fig. 17 and Table 1 that the hysteresis curve of strong axis loading is plumper than that of weak axis loading. In addition, $P, K$ of each
hysteresis loop, $h_{e}$ and $r$ of strong axis loading are larger than those of weak axis loading. Compared to specimens rch5, rch6, the $D I$ of rch1, rch3 are decreased from $4.83,2.48$ to $4.54,2.26$, decreased by $6.0 \%$, $8.9 \%$. The difference between $D I$ of them is not significant, indicating that the loading direction has little influence on the ductility of rectangular SCFT specimens.


(d) Energy dissipation

(e) Residual deformation

Fig.16. Influence of loading direction on hysteresis behavior when $\boldsymbol{n}$ is $\mathbf{0 . 2}$


(d) Energy dissipation

(e) Residual deformation

Fig.17. Influence of loading direction on hysteresis behavior when $\boldsymbol{n}$ is $\mathbf{0 . 7}$

### 4.7. Effect of height-length ratio $(L / B)$

Fig. 18 compares the influence of different height-length ratios $(L / B)$ on the hysteresis curve of specimens. As seen from Fig. 18 and Table 1, the higher the $L / B$ is, the less the $P$ and the $K_{1}$ are, the faster the horizontal load decreases, the worse the seismic behavior and the hysteresis loops are slightly pinched. Compared to sch6, the $L / B$ of sch4 and sch11 are increased from 4.5 to $7.5,10$, increased by $66.7 \%$, $122.0 \%$, respectively. But the $D I$ decreases from 4.71 to $4.38,4.08$, decreases by $7.0 \%, 13.4 \%$, respectively. Similarly, compared to sch7, the $L / B$ of sch 12 is increased from 7.5 to 10 , increased by $33.3 \%$, respectively. The $D I$ decreases from 2.85 to 2.63 , decreases by $7.7 \%$. What's more, compared to rch1, rch2 and rch7, the $L / B$ of rch8, rch9 and rch6 are increased from $5,5,5$ to $6.7,6.7,7.5$, increased by $34.0 \%, 34.0 \%, 50 \%$ respectively. The $D I$ decreases from $4.54,4.27,2.75$ to $4.12,3.85,2.47$, decreases by $9.3 \%, 9.8 \%, 10.2 \%$. It can be found that when the $L / B$ increases significantly, the $D I$ of the rectangular SCFT decreases very finitely.


(d) rch2 and rch9

(e) rch7 and rch6

Fig.18. Influence of height-length ratio on hysteresis curve

### 4.8. Effect of length-width ratio (B/D)

Fig. 19 and Fig. 20 compare the effect of two length-width ratios $B / D=1$ (square SCFT) and $B / D=1.5$ (rectangular SCFT) on hysteresis curve and energy dissipation, respectively. It is indicated from Fig. 19, Fig. 20 and Table 1 that when $B / D=1.5$, both the $K$ and the $P$ are larger. But the hysteresis loops demonstrate slightly pinched. On the contrary, when $B / D=1.0$, both the $K$ and the $P$ are smaller. But the hysteresis loops are plumper without pinched. At the same displacement, $h_{e}$ of square SCFT is greater than that of rectangular SCFT which indicates that the energy dissipation capacity of square SCFT is superior to that of rectangular SCFT. Compared to square SCFT rch3, sch10, sch11 and sch12, the $D I$ of rch2, rch8, rch9 and rch10 are decreased from $4.38,4.62,4.08$ and 3.63 to $4.27,4.22,3.85$ and 3.39 , decreased by $2.5 \%$, $8.7 \%, 5.6 \%$ and $6.6 \%$ respectively. It states that the influence of length-width ratio on the $D I$ of these rectangular SCFT is tiny.

(a) sch4 and rch2

(b) sch10 and rch8

(d) sch12 and rch10

Fig.19. Influence of length-width ratio on hysteresis curve


Fig.20. Influence of length-width ratio on equivalent viscous damping index

### 4.9. Effect of axial compression ratio ( $\boldsymbol{n}$ )

Fig.s 21, 22, 23, 24 compared the effects of different $n$ on the hysteretic curves, stiffness degradation, $D I$ and $h_{e}$. With the increase of $n$, the initial stiffness of the specimens is generally increased, but descends steeper and the $D I$ is obviously reduced. The $D I$ of rectangular SCFT with small $n$ or medium $n$ is generally larger than 3 , indicating that the ductility of the specimens is good and can meet the seismic design requirements. Among all specimens, the $D I$ of rch3 is the least, which is 2.26 due to its low $\rho_{\text {sa }}$ and
high value of $n(n=0.7)$. Based on the "General yield bending moment method", the failure displacement $\Delta_{u}$ of rch3 is 26.0 mm . Accordingly, the maximum displacement angle $\left(\Delta_{u} / L\right)$ is $1 / 58$, which cannot meet the limit value $1 / 50$ of relevant standards [28,29]. Therefore it is necessary to increase the $\rho_{\mathrm{sa}}$ and conduct the corresponding experimental study. When $n$ increases from 0 to 0.4 or from 0.2 to 0.4 , the $P$ of the specimens is generally increased. However, when $n$ is further increased to 0.6 or 0.7 , the $P$ is decreased, while the $K_{1}$ of the specimen is generally increased. With the increase of the $n$, the hysteresis loop is plumper and $h_{e}$ is generally increased, indicating that the energy dissipation capacity is enhanced. For reinforced concrete members in seismic field, the $h_{e}$ values is approximately $0.1 \sim 0.2$ [30]. By contrast, the $h_{e}$ values of the SCFT specimens in this test range from 0.15 to 0.4 , which demonstrate that the energy dissipation capacity of SCFT is better than that of reinforced concrete members.


Fig.21. Influence of $\boldsymbol{n}$ on hysteresis curve


(d) rch1, rch2 and rch3
(e) rch5 and rch6
(f) rch8, rch9 and rch10

Fig.22. Influence of $\boldsymbol{n}$ on stiffness degradation

(a) DI-n of square specimens (b) DI-n of rectangular specimens

Fig.23. DI-n relationship curve


Fig.24. Influence of $\boldsymbol{n}$ on equivalent viscous damping index

### 4.10. Effect of sliding support

Fig. 25 compares the effect of sliding support on the hysteresis behavior when $n$ is 0.4 and the stirrups range is $200 \mathrm{~mm}(B)$. Because the friction between the jack and the top plate is eliminated due to the sliding support, the skeleton curve of sch3 declines more gently and its stiffness degrade more slowly than sch5.

Also, we can see from Table 1 that the $D I$ of sch3 increased from 3.35 to 3.74 , increased by $11.6 \%$. However, friction leads the $K_{1}$ of sch5 increase from $4.11 \mathrm{kN} / \mathrm{mm}$ to $4.96 \mathrm{kN} / \mathrm{mm}$ and the $P$ of sch5 increased from 61.02 kN to 66.81 kN , increased by $20.7 \%$ and $9.5 \%$, respectively. In addition, the hysteresis curve of sch 5 is plumper and its maximum $h_{e}$ increased from 0.23 to 0.34 , increased by $47.8 \%$. The above results contribute that the impact of friction cannot be ignored in the test, and the sliding support can ensure the experimental data more accurate.


(d) Energy dissipation

(e) Residual deformation

Fig.25. Influence of sliding support on hysteresis behavior

## 5. Conclusions

This paper presents a pseudo-static experimental study on the seismic behavior of stirrup-confined concrete-filled rectangular steel tubular columns. The experimental program consists of 26 specimens with consideration of with or without stirrups, height of terminal stirrup region, equivalent stirrup ratio, loading direction, and axial compression ratio etc. Based on the results of failure mode, strain ratio, ultimate bearing capacity ductility, stiffness degradation, energy dissipation, and residual deformation, the following conclusions can be drawn:
(1) Under axial pressure and horizontal low cyclic load, the specimens are failed mainly by buckling of the steel tubes, crush of the core concrete and fracture of the stirrups. The equivalent viscous damping index of the specimens ranges from 0.15 to 0.4 , which demonstrates that they have good seismic energy dissipation capacity.
(2) The maximum strain ratio of typical specimens is more than 0.5 , showing that the steel tube exerts a good confinement on the core concrete. The axial pressure can increase the confining effect of steel tube to the core concrete. In addition, the stirrups can directly confine the core concrete, and reduce the maximum strain ratio of the steel tube.
(3) When the axial compression ratio is larger, the bidirectional stirrups can delay local buckling of steel tube, improve the confinement effect on the core concrete effectively, and thus increase the ultimate bearing capacity and ductility index, so as to significantly improve the seismic behavior of the rectangular SCFT columns. At the same axial compression ratio, increasing height of terminal stirrup region or
increasing equivalent stirrup ratio can also effectively improve the seismic behavior of the specimens.
(4) The ultimate bearing capacity of the rectangular SCFT of strong axis loading is distinctly greater than that of weak axis loading, but there is no obvious difference between their ductility. When the height-length ratio and length-width ratio increases, the ductility $D I$ decreases very limited.

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