Experimental Study of H-Shaped Honeycombed Stub Columns with Rectangular Concrete-Filled Steel Tube Flanges Subjected to Axial Load

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The behavior of H-shaped honeycombed stub columns with rectangular concrete-filled steel tube flanges (STHCCs) subjected to axial load was investigated experimentally. A total of 16 specimens were studied, and the main parameters varied in the tests included the confinement effect coefficient of the steel tube ($\xi$), the concrete cubic compressive strength ($f_{cu}$), the steel web thickness ($t_2$), and the slenderness ratio of specimens ($\lambda_s$). Failure modes, load-displacement curves, load-strain curves of the steel tube flanges and webs, and force mechanisms were obtained by means of axial compression tests. The parameter influences on the axial compression bearing capacity and ductility were then analyzed. The results showed that rudder slip diagonal lines occur on the steel tube outer surface and the concrete-filled steel tube flanges of all specimens exhibit shear failure. Specimen load-displacement curves can be broadly divided into elastic deformation, elastic-plastic deformation, and load descending and residual deformation stages. The specimen axial compression bearing capacity and ductility increase with increasing $\xi$, and the axial compression bearing capacity increases gradually with increasing $f_{cu}$ whereas the ductility decreases. The ductility significantly improves with increasing $t_2$, whereas the axial compression bearing capacity increases slightly. The axial compression bearing capacity decreases gradually with increasing $\lambda_s$ whereas the ductility increases. An analytical expression for the STHCC short column axial compression bearing capacity is established by introducing a correction function ($w$), which has good agreement with experimental results. Finally, several design guidelines are suggested, which can provide a foundation for the popularization and application of this kind of novel composite column in practical engineering projects.

1. Introduction

Conventional multistory light steel structures have been widely applied for construction and reconstruction projects [1–4]. Current structural systems are moving toward superhigh, long-span, and heavily loaded structures, and conventional light steel structures meet many new challenges such as small achievable span, low component bearing capacity, overall or local instability, complex joint construction, and stress concentration. Many studies [5–8] have been devoted to steel-concrete composite structures with short construction period and high bearing capacity. The steel-concrete composite frames not only can take advantage of steel structural properties but also enhance their stiffness and overall stability and overcome defects and shortcomings of conventional steel frame structures. This study proposes a novel assembled composite frame consisting of H-shaped honeycombed columns with rectangular
concrete-filled steel tube (CFST) flanges (STHCCs) and I-shaped honeycombed beams with rectangular CFST flanges [9–11]. The columns and beams are connected together through integral joints, as shown in Figure 1. To reduce the deflection of composite beams, prestressing tendons are arranged in the steel tubes of lower composite beam flanges, whose tension and anchorage are realized at the ends of the integral joint. The CFST sectional form is adopted as frame column flanges, where the concrete prevents local steel tube buckling, while the steel tube continuously restrains the concrete; hence, the concrete experiences three-dimensional compression, and interaction between the steel tube and concrete can be fully utilized. Honeycombed webs between double-limbed CFST flanges not only effectively connect independent double CFST columns but also greatly reduce composite member weights. Beam-column members and integral joints can be prefabricated in the factory, connected in the field, and then filled with concrete [12, 13]; therefore, there is no need to support the template. Pouring concrete of the lower floor does not affect upper steel member connections; hence, concrete can be poured continuously from the bottom to the top, and concrete maintenance time is not required, which can significantly shorten the construction period. The proposed composite structure frames are suitable for many new construction and renovation projects due to the large stiffness, high stability, and high bearing capacity.

Many studies have considered I-shaped steel beams with different flanges. Hassanein and Silvestre [14] investigated transverse distortion buckling of hollow steel tube flange beams (HTFPG) without stiffeners to obtain HTFPG buckling capacity, verified the corresponding analytical expression, and finally provided corresponding design recommendations for this kind of beams. Erdal and Saka [9] experimentally investigated the flexural properties of twelve full-scale noncomposite cellular beams subjected to a concentrated load and presented ultimate load carrying capacities of optimally designed steel cellular beams. Zhang et al. [15] derived a nondimensional critical moment expression for lateral torsional buckling of cantilever beams with tip lateral elastic braces based on stability theory. Zhang [16] investigated symmetric and antisymmetric lateral-torsional buckling (LTB) performance of prestressed steel I-beams and obtained analytical solutions for symmetric and antisymmetric LTB of prestressed I-beams. Based on tests and numerical analysis, Grilo et al. [17] proposed a new formula to determine the shear resistance of cellular beams for web-post buckling. Liu et al. [18] proposed an analytical solution for the lateral-torsional buckling (LTB) of steel girders with the CFST flange subjected to a concentrated load. Wang et al. [19] predicted flexural and shear yielding strength of short-span composite I-girder with concrete-filled tubular flanges and corrugated web. Studies on CFST composite columns have mainly focused on conventional CFST columns, latticed columns, and specially shaped columns with various sections. Lam and Gardner [20] investigated axial compression for 16 concrete-filled stainless steel tubes to identify parameter influences on column bearing capacity and established an analytical expression to predict the bearing capacity. The accuracy of the proposed expression was verified by comparison with predictions calculated by ACI and EC4. Sweidan et al. [21] investigated overall buckling of axially loaded I-shaped columns with circular web perforations and provided a simplified procedure for evaluating the buckling capacity of columns. Chen and Ou [22] studied CFST latticed column ultimate bearing capacity using ANSYS and proposed an expression based on a parametric analysis to predict ultimate bearing capacity for four-legged CFST latticed columns. Ellebody and Ghazy [23] experimentally studied fiber reinforced concrete-filled stainless steel tube columns subjected to axial and eccentric load and indicated that the EC4 code could accurately predict axial compressive bearing capacity for stainless steel tube concrete columns. Bao [24] proposed a practical method to calculate equivalent slenderness ratios for CFST latticed columns, and an analytical expression was derived to predict CFST latticed column bearing capacity with K-shaped joints. In order to simulate latticed columns reasonably, Fooladi and Banan [25] developed a practical super element based on the concept of finite element. Ekmekyapar and Al-Elwi [26] tested 18 short, medium, and long circular CFST columns and indicated that EC4 predictions exhibited significantly better agreement with test results, whereas AISC360-10 predictions were conservative for all specimens. Liu et al. [27] experimentally studied the effects of different parameters on column stability under axial compression for 6 L-shaped and 12 T-shaped CFST stub columns, and corresponding analytical expressions were proposed for stability bearing capacity based on experimental and finite element (FE) analysis results. Yang et al. [28] researched the seismic behavior of four-legged square CFST latticed members under lateral cyclic loading and revealed that the four-legged square CFST latticed specimens outperformed hollow steel ones in terms of hysteretic behavior. Wang et al. [29] experimentally investigated the axial compressive performance of novel L-shaped and T-shaped concrete-filled square steel tube (L/T-CFST) columns and established formulae to calculate the axial compressive strength and stability bearing capacity of the L/T-CFST columns accordingly. Shariati et al. [30] presented a numerical investigation into the behavior of built-up concrete-filled steel tube columns under axial compression and indicated that the raise of concrete strength with greater cross section led to a higher load bearing capacity compared to the steel tube thickness increment. However, most of the previous research studies were mainly concentrated on beams with different flanges and CFST columns with various sections, and it seems that there is seldom research reported on the performance of H-shaped concrete-filled columns with rectangular concrete-filled steel tube flanges (STHCCs).

The honeycombed steel webs connect the two independent rectangular CFST columns as a whole two-legged column, they can prevent the flanges from bending to both sides effectively, and it is similar to effect of oblique spars in the truss system. To explore mechanical behavior of STHCCs, our research group [12] has investigated the buckling behavior of STHCCs and
proposed eigenvalue and nonlinear buckling load expressions for this kind of composite columns. Based on this, this paper experimentally investigated axial compression of 16 STHCC stub columns, and the influences of different parameters on axial compression performance of the stub columns were studied. An analytical expression was derived to calculate ultimate bearing capacity of the stub columns based on the measured experimental statistics.

2. Test Overview

2.1. Specimen Design. To study STHCC axial compression, 16 specimens were designed with different hoop coefficient ($\xi$), concrete cubic compressive strength ($f_{cu}$), web thickness ($t_{2}$), and slenderess ratio ($\lambda_s$) parameter values (see Table 1), where the physical meaning of the variables is shown in Figure 2. Honeycomb webs were formed by opening holes on steel webs with a diameter-to-height ratio ($d/h_w$) of 0.7, and a space-between-holes-to-height ratio ($s/h_w$) of 0.3. Two steel cover plates were welded at the end of each specimen, and two rectangular pouring holes were arranged in the upper cover plate. To prevent local buckling of steel tubes and webs at the ends, 10 stiffening ribs were set amongst the steel tubes, webs, and cover plates. Figure 3 shows design and compositions of the specimens.

2.2. Specimen Production and Maintenance of Specimens. Rectangular steel tubes, steel webs, cover plates, and stiffening ribs were cut according to the specimen design, and rectangular holes were cut in symmetrical locations in the upper cover plates. The rust was removed from rectangular steel pipes, steel webs, and cover plates surfaces after cutting; then, these components were welded together (see Figures 4(a) and 4(b)). The H-shaped steel columns with rectangular steel tube flanges were placed on the vibrating platform, and concrete was poured through the rectangular holes. When the assembly was vibrated until floating slurry appeared, it is proved that the columns were completely filled and compacted. Finally, the concrete surface was pressed and leveled. Three standard cubic blocks ($150 \times 150 \times 150$ mm) were also poured for each concrete mixture to test compressive strength. After these blocks were demolded, they were maintained under the same conditions as specimens in the laboratory. The images of the final specimens are shown in Figure 4(c).

2.3. Material Mechanical Properties

2.3.1. Steel. Specimens were reasonably designed in accordance with the relevant regulations [32]. The experiment included 6 kinds of steel thicknesses (1.7, 2.3, 3.8, 6, 8, and 12 mm), and three identical samples were tested for each thickness, i.e., 18 samples. Table 1 lists the measured average yield strength of tensile samples ($f_{yt}$ and $f_{yw}$).

2.3.2. Concrete. Four types of concrete (A, B, C, and D) were prepared with different strengths [33]. Table 2 lists the mix proportions between cement, fine, and coarse aggregate (P. O. 42.5 Portland cement, river sand, and gravel, respectively) and the corresponding average cubic compressive strength ($f_{cu}$).

2.4. Loading Scheme and Measurement. Axial compression tests for all specimens were performed using a four column pressure tester (see Figure 5) in the Key Laboratory of Disaster Prevention and Mitigation Engineering and Protection Engineering of Heilongjiang Province. Specimens were placed on the pressure tester and leveled by the SHEFFIELD-S078009 level ruler. Automatic double-controlled loading was adopted. Preloading and unloading were performed during the preloading stage. Normal loading began after preloading with a rate of 0.5 kN/s in the initial stage. When vertical load reached 50% of the predicted peak load, the loading mode was changed to displacement loading with a loading rate of 0.02 mm/s. Axial load and displacement were automatically collected using computer software throughout the loading process. Resistance strain gauges (RSGs) were placed near the central section of the specimen flanges and webs, and DH3818Y static resistance
strain devices were used to collect the steel strains. Figure 6 displays the residual strain gauge (RSG) layout at different heights for the specimens.

Table 1: Main parameters of specimens.

| Specimens | Dimensions $h_w \times h_1 \times b \times t_1 \times t_2$ $(mm^3)$ | $l$ (mm) | $\lambda_s$ | $f_yf$ (MPa) | $f_{yw}$ (MPa) | $\xi$ | $f_{cu}$ (MPa) |
|-----------|------------------------------------------------|---------|-----------|------------|------------|------|-------------|
| STHCC-1   | $100 \times 50 \times 100 \times 1.7 \times 06$ | 370     | 12.82     | 269        | 321        | 0.60 | 49.50       |
| STHCC-2   | $100 \times 50 \times 100 \times 1.7 \times 06$ | 370     | 12.82     | 269        | 321        | 0.60 | 49.50       |
| STHCC-3   | $100 \times 50 \times 100 \times 2.3 \times 06$ | 370     | 12.82     | 282        | 321        | 0.88 | 49.50       |
| STHCC-4   | $100 \times 50 \times 100 \times 3.8 \times 06$ | 370     | 12.82     | 286        | 321        | 1.60 | 49.50       |
| STHCC-5   | $100 \times 50 \times 100 \times 3.8 \times 06$ | 370     | 12.82     | 286        | 321        | 1.60 | 49.50       |
| STHCC-6   | $100 \times 50 \times 100 \times 2.3 \times 06$ | 370     | 12.82     | 282        | 321        | 0.82 | 53.17       |
| STHCC-7   | $100 \times 50 \times 100 \times 2.3 \times 06$ | 370     | 12.82     | 282        | 321        | 0.66 | 65.60       |
| STHCC-8   | $100 \times 50 \times 100 \times 2.3 \times 06$ | 370     | 12.82     | 282        | 321        | 0.78 | 55.09       |
| STHCC-9   | $100 \times 50 \times 100 \times 1.7 \times 08$ | 370     | 12.82     | 269        | 325        | 0.60 | 49.50       |
| STHCC-10  | $100 \times 50 \times 100 \times 1.7 \times 12$ | 370     | 12.82     | 269        | 331        | 0.60 | 49.50       |
| STHCC-11  | $100 \times 50 \times 100 \times 2.3 \times 06$ | 270     | 12.82     | 282        | 321        | 0.88 | 49.50       |
| STHCC-12  | $100 \times 50 \times 100 \times 2.3 \times 06$ | 470     | 16.28     | 282        | 321        | 0.88 | 49.50       |
| STHCC-13  | $100 \times 50 \times 100 \times 1.7 \times 06$ | 270     | 9.35      | 269        | 321        | 0.60 | 49.50       |
| STHCC-14  | $100 \times 50 \times 100 \times 1.7 \times 06$ | 470     | 16.28     | 269        | 321        | 0.60 | 49.50       |
| STHCC-15  | $100 \times 50 \times 100 \times 3.8 \times 06$ | 270     | 9.35      | 286        | 321        | 1.60 | 49.50       |
| STHCC-16  | $100 \times 50 \times 100 \times 3.8 \times 06$ | 470     | 16.28     | 286        | 321        | 1.60 | 49.50       |

Note. $h_w$, $h_1$, $b$, $t_1$, and $t_2$ are the web width, flange steel tubes’ height, width, and thickness, and web thickness, respectively. $l$ is specimen length. $\lambda_s$ is the slenderness ratio, i.e., $\lambda_s = \sqrt{3}/b$, and $f_yf$ and $f_{yw}$ are the measured average yield strength for flange and web steel, respectively. $f_{cu}$ is the measured average cubic compressive strength for concrete. $\xi$ is the hoop coefficient, i.e., $\xi = A_{f,y}/A_{f,yk}$. The physical meaning of these variables is explained in the study by Yang and Han [31].

3. Test Results and Analysis

3.1. Test Phenomena and Failure Modes. Figure 7 shows the specific failure modes for the 16 short columns subjected to axial compression. The failure development process is basically similar in the range of test parameters.

(i) During the initial loading, vertical load was small and there was no significant surface change for all specimens. Steel strain increased linearly, and load-displacement ($L-D$) and load-strain ($L-S$) curves increased linearly.

(ii) With increasing vertical load, the attachment to the steel tube surface gradually fell off, and a rudder slip diagonal line formed on the outer surface of the steel tubes. Slight outward bulging appeared at the W and E sides (see Figure 5) of both ends of some specimens. Specimens began to show nonlinear characteristics, with $L-D$ curves gradually deviating from the original linear trajectory. When load reached 80% of peak load, slight bulges appeared in the middle of some specimens. With further increasing of vertical displacement, swelling in the middle of the specimens increased gradually, accompanied by the sound of cracking concrete.

(iii) When the specimens reached ultimate bearing capacity, vertical load reduced gradually and the specimens showed good ductility. The N and W side bulges of some specimens increased gradually, and S and N sides’ multiple bulges occurred in the upper, middle, and lower sections as vertical load reduced. The flanges of both sides of the specimens bent outward gradually, the webs shrunk, and honeycomb holes gradually evolved into ellipses. With further increasing of vertical displacement, bulges on the flanges gradually connected to form overall bulging for the section.
(iv) CFST flanges showed shear failure characteristics for all specimens.

(v) Comparing web failure modes of the specimens, 370 and 470 mm specimens showed web deformation developed along the same plane as the sides of the honeycombs, and the honeycombs gradually became elliptical without local buckling. However, for 270 mm length specimens, web deformations not only developed along the same plane to the two sides of the honeycombs but also appeared to buckle outward slightly.

3.2. Load-Displacement Curves. Figure 8 shows the L-D curves for the 16 specimens. It is seen that the curves can be broadly divided into four stages.

Stage 1: elastic deformation (OA). The L-D curves increase linearly with basically constant slope. For specimens STHCC-14–STHCC-16, nonlinearity appears in the initial stage of L-D curves, which is mainly due to poor compaction between the loading device and specimens. This error has no effect on the overall process analysis for the specimens.

Stage 2: elastic-plastic deformation (AB). The steel tubes enter the elastic-plastic state, and the restrained concrete shows significant nonlinearity. L-D curves significantly deviate from linearity with increasing slopes. Finally, the specimens reach ultimate load, i.e., peak bearing capacity (point B).

Stage 3: load reduction (BC). After peak load, the load in the L-D curves reduces and the reduction rate is very slow. All specimens show good plasticity and good

Table 2: Concrete compositions.

| Types | Mix proportion Cement : sand : gravel : WRA : SF | Water ash ratio w/c | Cement content (kg·m⁻³) | $f_{cu}$ (MPa) |
|-------|-----------------------------------------------|---------------------|--------------------------|--------------|
| A     | 1 : 1.111 : 2.062 : 0 : 0                      | 0.380               | 487                      | 49.50        |
| B     | 1 : 1.020 : 2.062 : 0.010 : 0                   | 0.288               | 548                      | 53.17        |
| C     | 1 : 1.469 : 2.869 : 0.0293 : 0.0995             | 0.272               | 437                      | 65.60        |
| D     | 1 : 1.660 : 1.871 : 0.0411 : 0.1371             | 0.295               | 474                      | 55.69        |

Note. WRA = water reducing agent; SF = silica fume.
displacement ductility. After passing the inflection point (point C), the $L-D$ curves change from convex to concave.

Stage 4: residual deformation (CD). Specimen deformation increases, and bulging around the steel tubes connects together, while the core concrete is still restrained by the steel tubes, and the specimens retain significant bearing capacity. Finally, the specimen is destroyed.

3.3. Steel Tube Flange Load-Strain Curves. Due to the symmetrical cross section of the STHCC specimens, the deformation trends are basically consistent for left and right flanges. Figure 9 shows only the longitudinal and transverse load-strain ($L-S$) curves for the right flanges (see Figures 9 ($a_1-p_1$) and ($a_2-p_2$)), respectively.

Initial loading $L-S$ curves are linear for all specimens. With increasing load, the slope gradually decreases and the steel tube enters the elastic-plastic stage. After peak load, the steel tube strain rate gradually accelerates, and bulging around the steel tubes is connected together in many places. The $L-S$ curves of the steel tubes become close to horizontal, and the steel tubes have entered the complete plastic state.

3.4. Steel Web Load-Strain Curves. Figure 10 shows longitudinal and transverse $L-S$ curves for the steel webs. The web strain is relatively complex and smaller compared with flange strain, i.e., web deformation is smaller than that of flanges under load; hence, local buckling does not occur for the webs during loading, maintaining stable vertical compression deformation. The rectangular CFST flanges mainly bear load for STHCCs, whereas the honeycombed webs mainly connect the two limbs of the flanges. The web itself has little effect on improving bearing capacity for this kind of composite short column.

3.5. Specimen Force Mechanism. When the specimens are subjected to longitudinal axial pressure, $P$, longitudinal strain, $\varepsilon_3$, will be generated in the steel tubes and concrete.
Figure 7: Specimen failure modes. (a) STHCC-1. (b) STHCC-2. (c) STHCC-3. (d) STHCC-4. (e) STHCC-5. (f) STHCC-6. (g) STHCC-7. (h) STHCC-8. (i) STHCC-9. (j) STHCC-10. (k) STHCC-11. (l) STHCC-12. (m) STHCC-13. (n) STHCC-14. (o) STHCC-15. (p) STHCC-16.
The transverse strain that steel tubes and concrete ($\epsilon_{1s}$ and $\epsilon_{1c}$ respectively) generate can be expressed as

$$\epsilon_{1s} = \mu_s \epsilon_1,$$  \hspace{.25in} (1)

$$\epsilon_{1c} = \mu_c \epsilon_1,$$  \hspace{.25in} (2)

where $\mu_s$ and $\mu_c$ are Poisson’s ratio for steel and concrete, respectively.

Initially, $\mu_c$ is lower than $\mu_s$ for the CFST flanges. When steel longitudinal compressive stress reaches its proportional limit, i.e., $\delta_3 \approx f_p / \mu_s$, $\mu_c \approx \mu_s$. As $P$ increases, steel stress exceeds its proportional limit, i.e., $\delta_3 > f_p / \mu_s > \mu_c$. From (1) and (2), the corresponding concrete strain exceeds that of steel, i.e., $\epsilon_{1c} > \epsilon_{1s}$, and expansion of the concrete core exceeds that of the steel tubes; hence, the concrete core is restrained by the steel tubes, and its lateral expansion is hindered. Figure 11(a) shows the interaction force generated between the steel tube and concrete core. Both the steel tubes and concrete core are under three-directional force (see Figure 11(b)). Since the steel tubes bulge outwards under the concrete and steel interaction (see Figure 12(a)), the hoop force significantly reduces in the middle of the edge of steel tubes (see Figure 13(a)). Circumferential shear stress ($r$) and normal stress pointing to the web honeycomb are also produced when the steel webs are subjected to $P$, as shown in Figure 13(b). Steel tube flanges’ stability is improved due to the support provided by the concrete core, allowing the steel yield strength to be fully utilized.

After peak load, the specimens bulge in many places and the bulges are connected together when displacement increases, as shown in Figure 12(b). The honeycombed steel...
Figure 9: Continued.
Figure 9: Steel flange load-strain (L-S) curves.

Figure 10: Continued.
Figure 10: Steel web load-strain ("L-S") curves.  
(a) STHCC-1-5, web 9. (b) STHCC-1-5, web 10.  
(c) STHCC-6-8, web 9. (d) STHCC-6-8, web 10.  
(e) STHCC-9-10, web 9. (f) STHCC-9-10, web 10.  
(g) STHCC-11-16, web 5 or 9. (h) STHCC-11-16, web 6 or 10.

Figure 11: 3D steel and concrete stress.  
(a) Steel tube.  
(b) Steel and concrete.

Figure 12: Local bulging and connection.  
(a) Local bulging.  
(b) Bulging connection.
webs connect two rectangular CFST columns as a whole; hence, the webs prevent the flanges from bending to both sides, which is similar to the effect of oblique spars in truss systems, thereby significantly improving ultimate bearing capacity of the composite short columns.

4. Parameter Analysis

4.1. Hoop Coefficient. Figure 14 shows L-D curves of specimens for different $\xi$, and Table 3 lists the ultimate bearing capacity ($N^T_u$) with different $\xi$. Stub column ductility is usually expressed using the ductility coefficient, where the larger coefficient implies improved ductility and vice versa.

$kV_h$ is paper analyzed STHCC specimen ductility using the displacement ductility coefficient:

$$\mu = \frac{\Delta_u}{\Delta_y}$$  \hspace{1cm} (3)

where $\Delta_y$ and $\Delta_u$ are the yield and ultimate displacements obtained from the L-D curves for the specimens.

The energy equivalent method [34] is used to determine the equivalent yield point, as shown in Figure 15. We can obtain points $A$ and $B$ by making area $(OAO) = area (ABCA)$, i.e., $S_{OAO} = S_{ABCA}$.

The projection of point $B$ onto the $x$ axis intersects with the L-D curves at point $E$, which is the equivalent yield point. The corresponding load and displacement are the yield load ($P_y$) and yield displacement ($\Delta_y$), respectively. The ultimate displacement ($\Delta_u$) is the displacement corresponding to the load falling to the nominal load limits, and we regarded 85% of the ultimate load as the nominal limit load. Table 3 lists $\mu$ for specimens with different $\xi$.

When $\xi$ increases from 0.60 to 0.88, STHCC-3 $N^T_u$ increases by 22.4% and 16.6%, respectively (Figure 14 and Table 3), compared to STHCC-1 and STHCC-2. When $\xi$ increases from 0.88 to 1.60, STHCC-4 and STHCC-5 $N^T_u$ are 35.4% and 36.7% higher than that of STHCC-3, respectively. When $\xi$ increases from 0.60 to 1.60, specimen $\mu$ increases significantly from 2.47 to 5.66. To improve ultimate bearing capacity and ductility, $\xi$ may be increased appropriately under the condition that the steel ratio of the column meets the design requirements of STHCC columns.

4.2. Concrete Cubic Compressive Strength. Figure 16 shows L-D curves for 4 specimens with different $f_{cu}$ and Table 4 lists corresponding $N^T_u$ and $\mu$. Specimen $N^T_u$ increases by 23.5% as $f_{cu}$ increases from 49.50 to 53.17 MPa, and 9.9% as $f_{cu}$ increases from 53.17 to 65.60 MPa, whereas $\mu$ reduces from 3.36 to 1.90 as $f_{cu}$ increases from 49.50 to 65.60. Thus, enhancing concrete strength can improve ultimate bearing capacity of the composite stub columns, while reducing ductility.

4.3. Steel Web Thickness. Figure 17 shows L-D curves for specimens with different $t_2$, and Table 5 lists corresponding $N^T_u$ and $\mu$. Specimen $N^T_u$ increases from 706.12 and 741.21 to
763.17 kN (8.1% and 3.0%, respectively) as $t_2$ increases from 6 to 8 mm. However, specimen $N_{u1}^T$ increases from 763.17 to 798.26 kN (4.6%), and $\mu$ increases from 2.47 to 5.84 (136.4%) as $t_2$ increases from 6 to 12 mm. Thus, increasing web thickness can effectively improve composite stub column ductility, while the contribution to increasing bearing capacity is not significant. Therefore, appropriately increasing web thickness will improve composite stub columns’ stability and vertical seismic performance.

4.4. Slenderness Ratio. Figure 18 shows $L$-$D$ curves for specimens with different $\lambda_s$, and Table 6 lists corresponding $N_{u1}^T$ and $\mu$. Specimen $N_{u1}^T$ decreases by 11.3% as $\lambda_s$ increases from 9.35 to 16.28 (Figure 18(a)), from 781.37 to 716.92 kN (8.3%) as $\lambda_s$ increases from 9.35 to 16.28 (Figure 18(b)), and decreases from 1261.72 to 1076.86 kN (14.6%) when $\lambda_s$ increases from 9.35 to 16.28 (Figure 18(c)). Thus, specimen ultimate bearing capacity decreases with increasing slenderness ratio. Table 6 and Figure 19 also show that specimen $\mu$ increases correspondingly with increasing $\lambda_s$.

5. STHCC Ultimate Bearing Capacity

$N_{u1}^T$ for STHCC composite stub columns subjected to axial load can be obtained from test results of 16 STHCC specimens with different parameters. To consider influences of $\xi$, $f_{cu}$, $t_2$, and $\lambda_s$ on ultimate bearing capacity of composite stub columns, an analytical expression ($N_{u1}$) is constructed by superimposing bearing capacity of rectangular CFST columns and webs based on the expression for bearing capacity of ordinary rectangular CFST columns:

$$N_{u1}^T = 2(\alpha \lambda_s^2 + b \xi + c)A_{sc}(1.18 + 0.85\xi)f_{ck} + k_yf_{yw}h_w t_2,$$

$$a = \frac{1 + (35 + 2\lambda_p - \lambda_b) \cdot e}{(\lambda_p - \lambda_b)^2},$$

$$b = e - 2\alpha\lambda_p,$$

$$c = 1 - a\lambda_b^2 - b\lambda_b,$$

$$d = \left[13500 + 4810 \ln \left(\frac{235}{f_{yw}}\right)\right] \left(\frac{25}{f_{ck} + 5}\right)^{0.3} \left(\frac{\alpha}{0.1}\right)^{0.05},$$

$$e = \left(\frac{\lambda_p + 35}{5}\right)^3,$$

where $A_{sc} = h_1 b$, $\alpha = A_{sc}/A_s$, $\lambda_s = 2\sqrt{3}s/h_1$, $\lambda_p = 1811/\sqrt{f_{yw}}$, $\lambda_b = \frac{\pi}{2}\sqrt{(220\xi + 450)/([0.85\xi + 1.18]f_{ck})}$, $\lambda_s$ is the slenderness ratio of single CFST flange for specimens, $A_{sc}$ is the cross-sectional area, $\xi$ is the hoop coefficient, $f_{ck}$ is the standard concrete axial compressive strength, $f_{yw}$ is the steel flange yield strength, $f_{yw}$ is the steel web yield strength, $\alpha$ is...
Table 6: Specimen ultimate bearing capacity and ductility for different $\lambda_s$.

| Specimens | $t_1$ (mm) | $l$ (mm) | $\lambda_s$ | $N_u$ (kN) | $\Delta_y$ (mm) | $\Delta_u$ (mm) | $\mu$  |
|-----------|------------|----------|-------------|-------------|----------------|----------------|-------|
| STHCC-1   | 1.70       | 370      | 12.82       | 706.12      | 2.23           | 5.51           | 2.47  |
| STHCC-2   | 1.70       | 370      | 12.82       | 741.21      | 2.06           | 5.13           | 2.49  |
| STHCC-3   | 2.30       | 370      | 12.82       | 864.59      | 1.64           | 5.52           | 3.36  |
| STHCC-4   | 3.80       | 370      | 12.82       | 1170.37     | 2.81           | 14.46          | 5.15  |
| STHCC-5   | 3.80       | 370      | 12.82       | 1182.15     | 1.83           | 10.36          | 5.66  |
| STHCC-11  | 2.30       | 270      | 9.35        | 948.52      | 1.69           | 5.49           | 3.25  |
| STHCC-12  | 2.30       | 470      | 16.28       | 841.31      | 1.31           | 6.82           | 5.21  |
| STHCC-13  | 1.70       | 270      | 9.35        | 781.37      | 1.29           | 3.12           | 2.42  |
| STHCC-14  | 1.70       | 470      | 16.28       | 716.92      | 1.67           | 6.16           | 3.69  |
| STHCC-15  | 3.80       | 270      | 9.35        | 1261.72     | 2.26           | 11.07          | 4.90  |
| STHCC-16  | 3.80       | 470      | 16.28       | 1076.86     | 2.02           | 12.44          | 6.16  |

Figure 18: Specimen load-displacement curves for different $\lambda_s$. (a) $t_1 = 2.3$ mm. (b) $t_1 = 1.7$ mm. (c) $t_1 = 3.8$ mm.

Figure 19: Specimen $\mu$ for different $\lambda_s$. 

# Advances in Civil Engineering
Table 7: $N^C_{u1}$ and $N^T_{u1}$ of 16 specimens.

| Specimens  | $h_w \times h_t \times b \times t_1 \times t_2$ (mm$^3$) | $\xi$ | $\lambda_i$ | $f_{cu}$ (MPa) | $N^C_{u1}$ (kN) | $N^T_{u1}$ (kN) | $|N^C_{u1} - N^T_{u1}| / N^C_{u1}$ (%) | $|N^C_{u1} - N^T_{u1}| / N^T_{u1}$ (%) |
|------------|-----------------------------------------------|------|------------|----------------|----------------|----------------|--------------------------------|--------------------------------|
| STHCC-1    | $100 \times 50 \times 100 \times 1.7 \times 06$ | 0.6  | 12.82      | 49.50         | 706.12        | 602.65        | 14.65                          | 4.27                           |
| STHCC-2    | $100 \times 50 \times 100 \times 1.7 \times 06$ | 0.6  | 12.82      | 49.50         | 741.21        | 602.65        | 18.69                          | 0.67                           |
| STHCC-3    | $100 \times 50 \times 100 \times 2.3 \times 06$ | 0.8  | 12.82      | 49.50         | 864.59        | 683.77        | 20.91                          | 1.43                           |
| STHCC-4    | $100 \times 50 \times 100 \times 3.8 \times 06$ | 1.6  | 12.82      | 49.50         | 1170.37       | 878.33        | 24.95                          | 1.57                           |
| STHCC-5    | $100 \times 50 \times 100 \times 3.8 \times 06$ | 1.6  | 12.82      | 49.50         | 1182.15       | 878.33        | 25.70                          | 0.55                           |
| STHCC-6    | $100 \times 50 \times 100 \times 2.3 \times 06$ | 0.8  | 12.82      | 53.17         | 1067.75       | 711.39        | 33.37                          | 7.95                           |
| STHCC-7    | $100 \times 50 \times 100 \times 2.3 \times 06$ | 0.8  | 12.82      | 65.60         | 1173.69       | 798.97        | 31.93                          | 0.78                           |
| STHCC-8    | $100 \times 50 \times 100 \times 2.3 \times 06$ | 0.7  | 12.82      | 55.69         | 1086.86       | 729.67        | 32.86                          | 4.52                           |
| STHCC-9    | $100 \times 50 \times 100 \times 1.7 \times 08$ | 0.6  | 12.82      | 49.50         | 763.17        | 633.04        | 17.05                          | 0.46                           |
| STHCC-10   | $100 \times 50 \times 100 \times 1.7 \times 12$ | 0.6  | 12.82      | 49.50         | 798.26        | 694.64        | 12.98                          | 3.76                           |
| STHCC-11   | $100 \times 50 \times 100 \times 2.3 \times 06$ | 0.8  | 9.35       | 49.50         | 948.52        | 702.32        | 25.96                          | 0.69                           |
| STHCC-12   | $100 \times 50 \times 100 \times 2.3 \times 06$ | 0.8  | 16.28      | 49.50         | 841.31        | 664.59        | 21.01                          | 2.67                           |
| STHCC-13   | $100 \times 50 \times 100 \times 1.7 \times 06$ | 0.6  | 9.35       | 49.50         | 781.37        | 623.45        | 20.21                          | 3.04                           |
| STHCC-14   | $100 \times 50 \times 100 \times 1.7 \times 06$ | 0.6  | 16.28      | 49.50         | 716.92        | 585.30        | 18.36                          | 3.87                           |
| STHCC-15   | $100 \times 50 \times 100 \times 3.8 \times 06$ | 1.6  | 9.35       | 49.50         | 1261.72       | 900.24        | 28.65                          | 0.52                           |
| STHCC-16   | $100 \times 50 \times 100 \times 3.8 \times 06$ | 1.6  | 16.28      | 49.50         | 1076.86       | 855.59        | 20.55                          | 3.74                           |

Figure 20: Continued.
the steel content, and $k_s$ is the reduction factor of the webs calculated according to Zhang [34].

Table 7 lists calculated ultimate bearing capacity ($N_{u1}^C$) and experimental value ($N_u^C$) of composite short columns. $N_{u1}^C$ calculated by the superposition method exhibits larger error compared with the experimental value; hence, bearing capacity obtained by simple superposition is lower and conservative. Considering coupling effect of CFST flanges and honeycombed webs, this study proposes an analytical expression by means of introducing correction function ($\omega$) and influence factor ($\eta$) and modifies (4) as follows:

$$N_u = N_{u1} + \omega(\eta). \hspace{1cm}(10)$$

The experimental data (listed in Table 7) indicate that $\eta$ is proportional to $\xi$ and the $n^{th}$ power of $f_{cu}$, while inversely proportional to $\lambda_s$, i.e.,

$$\eta = \frac{\xi \times f_{cu}^n}{\lambda_s}. \hspace{1cm}(11)$$

where $n = 1, 2, 3, \ldots, 8$.

Figure 20 shows fitting curves for $\omega$ and $\eta$ when $n = 1, 2, \ldots, 8$. The fitted curves are consistent with experimental points when $n = 6, 7, \text{ and } 8$. It is seen from Figure 20 that the error is not significant for $n > 6$. Therefore, $n$ is determined to be 8:

$$\omega = -3.8 \times 10^{-17} \eta^2 + 2.5 \times 10^{-7} \eta - 20.5. \hspace{1cm}(12)$$

By substituting equation (12) into equation (10), the STHCC ultimate bearing capacity ($N_u$) is expressed as

$$N_u = 2(\alpha \lambda_s^2 + b \lambda_s + c)A_{sc}(1.18 + 0.85\xi)f_{ck} + k_s f_{yw} h_w \xi^2 \omega^2$$

$$- 3.8 \times 10^{-17} \left(\frac{\xi \times f_{cu}^n}{\lambda_s}\right)^2 + 2.5 \times 10^{-7} \left(\frac{\xi \times f_{cu}^n}{\lambda_s}\right) - 20.5. \hspace{1cm}(13)$$

Table 7 lists the calculated STHCC short column ultimate bearing capacity ($N_{u1}^C$) from equation (13). Figure 21
compares the experimental and calculated results before and after correction. There is relatively small error between calculated and experimental ultimate bearing capacity, which is sufficient for practical engineering. Therefore, the improved expression (equation (13)) is more reasonable to calculate STHCC short column ultimate bearing capacity than the classical superposition approach.

6. Conclusion

Axial compression of 16 STHCC stub specimens are experimentally investigated with regard to the hoop coefficient (ξ), concrete cubic compressive strength (fcu), web thickness (t2), and slenderness ratio parameters (λs). The experimental phenomena, failure modes, L-D curves, and flange and web L-S curves are obtained. The force mechanism is clearly defined for the specimens.

A rudder slip diagonal line forms on the outer surface of the specimens. CFST flanges show shear failure characteristics for all specimens, and the flanges bend outward gradually; meanwhile, the webs shrink, which lead to honeycomb circular holes gradually evolving into ellipses. The L-D curves for 16 specimens can be broadly divided into four stages: elastic deformation, elastic-plastic deformation, load reduction, and residual deformation. The L-S curves show that the flanges mainly bear load for STHCCs, whereas the honeycombed webs mainly connect the two limbs of the flanges. When the transverse strain of the core concrete exceeds that of the steel tubes, both the steel tubes and core concrete are under three-directional stress; hence, the interaction force is generated between the steel tubes and core concrete.

The influences of parameters on ultimate bearing capacity and ductility are systematically investigated. The results indicate that, with increasing ξ, the ultimate bearing capacity and the ductility of the specimens improve correspondingly. With increasing fcu, the ultimate bearing capacity of the specimens enhances accordingly; on the contrary, the ductility decreases gradually. With the increasing of t2, the ductility improves significantly, whereas the ultimate bearing capacity increases slightly. With the increasing of λs, the ultimate bearing capacity of the specimens decreases gradually; conversely, the ductility increases accordingly.

By means of introducing the correction function (ω) and the influence factor (η), an analytical expression calculating STHCC short column ultimate bearing capacity is established, which has good agreement with experimental results.

Data Availability

The data used to support the findings of this study are included within the article.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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