Buckling Analysis and Section Optimum for Square Thin-Wall CFST Columns Sealed by Self-Tapping Screws

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Two columns of thin-walled concrete-filled steel tubes (CFSTs), in which tube seams are connected by self-tapping screws, are axial compression tested and FEM simulated; the influence of local buckling on the column compression bearing capacity is discussed. Failure modes of square thin-wall CFST columns are, first, steel tube plate buckling and then the collapse of steel and concrete in some corner edge areas. Interaction between concrete and steel makes the column continue to withstand higher forces after buckling appears. A large deflection analysis for tube elastic buckling reflects that equivalent uniform stress of the steel plate in the buckling area can reach yield stress and that steel can supply enough designing stress. Aiming at failure modes of square thin-walled CFST columns, a B-type section is proposed as an improvement scheme. Comparing the analysis results, the B-type section can address both the problems of corner collapse and steel plate buckling. This new type section can better make full use of the stress of the concrete material and the steel material; this type section can also increase the compression bearing capacity of the column.

1. Introduction

When concrete filled steel tube (CFST) columns are under compression, the lateral expending of the concrete is constrained by the surrounding steel tube; this surrounding constraint can compress the concrete into a multidirectional compression state and increase the strength and deformation capacity of concrete, while the local buckling of the steel tube is restrained by the infill concrete. For this reason, CFST columns exhibit excellent structural performance [1–3]. Furthermore, in CFST columns, compared with reinforced concrete (RC) columns, all steel material is located at the perimeter of the cross section. Thus, the contribution of the steel section to flexural capacity can be maximized [4]. On the contrary, compared with RC frame structures, CFST column-steel beam frame structures would have no formwork and could be completely prefabricated [5]. This structural system can improve construction quality in rural areas where there is a lack of construction technology and provide the possibility of prefabrication and assembly for Chinese rural housing.

Since “New Rural Construction” was proposed by Chinese government in 2005, rural construction in China has been enhanced yearly. Historically, limited to certain economic means, most Chinese rural housing areas adopted masonry structures [6] because masonry houses have simplified construction [7] and good heat insulation [8]. However, brittleness and low bearing capacity [9] caused serious damage of rural masonry housing during earthquakes. Common earthquake damage phenomenon of masonry housing is inclined or “X”-shaped cracks appearing in walls between windows or spandrel wall, and structures often collapsed seriously [10], as shown in Figure 1. Since 2008, the year of the Wenchuan Earthquake, the seismic performance of rural housing has received increased attention in China, “Aseismatic Structure Atlas of Rural Housing” (DBJT13-58) [11] is promulgated by the Fujian Provincial Construction Department, and “Practical Illustration of Rural Housing’s Aseismatic Structure” [12] is published by the Architectural Design Institute of Zhejiang Province. Frame structural systems are suggested to replace masonry structures for new rural housing.

Most rural housing areas contain low-rise buildings, and according to “Code for design of concrete structures” (GB50010-2010) [13], the reinforcement ratio is suggested as...
0.6%–5%. Take an example of a commonly used RC column section in rural housings, the size is 400 mm × 400 mm. Usually, the sum of the reinforcement ratio and the stirrup ratio is approximately 2%. If all reinforcements and stirrups are steel tubes that have the same volume of steel materials and side length $b$ of the column section, the thickness of the steel tube is $t \approx 2$ mm, and $b/t = 200$. This type of CFST column is called thin-walled CFST, and it is different from columns in the current design specification CECS159 (China) [14], ACI-318 (U.S.) [15], AISC-360 (U.S.) [16], BS5400 (U.K.) [17], and AIJ (1997) [18], which aimed at using columns with steel pipe thickness greater than 6 mm.

Steel tubes of thin-walled CFST columns need to be cold formed and rolled, and tube junctions cannot be welded because of the thin thickness. Furthermore, the problem of local buckling of thin-walled CFST columns is more prominent. In this study, 2 thin-walled CFST columns in which tube seams are closed by self-tapping screws are axial compression tested and FEM simulated; the influence of local buckling to column compression bearing capacity is discussed.

2. Experimental Program

2.1. Test Specimens. All steel tubes are cold formed by Q345 steel, the filling concrete is C30, and the material properties are shown in Table 1. $E_c$ and $f_c$ are the compression module and compression strength of concrete, respectively; $E_s$ and $f_y$ are elastic module and yield strength of steel, respectively. The specimen with length $l = 650$ mm is named Z650 and that, with $l = 1200$ mm, is named Z1200. All column sections are $200$ mm × $200$ mm square, and the tube thickness is 1 mm. Each seam of the steel tube is connected by 2 team self-tapping screws. Spacing between screw teams is 60 mm in the horizontal direction and 50 mm in the vertical direction. Each screw is 20 mm in length and 5 mm in diameter. The manufacturing process of steel tubes is shown in Figure 2, and specimen sizes are shown in Figure 3.

2.2. Experimental Devices. A compression tester is used on the specimens with a 2000 kN applied load. Strain gauges are pasted on top, middle, and bottom parts of the column surfaces. Two dial indicators are arranged symmetrically to measure the total compressing deformations of the specimens. Experimental devices are shown in Figure 4. At the beginning, 50 kN is applied as a preloading stress; in formal loading, 50 kN is treated as the increment of each loading step, and the loading speed is controlled as 3.5~4.5 kN/s until specimens reach the maximum bearing capacity. Then, the control loading is changed in increments of 0.1 mm deformation in each step.

2.3. Test Phenomenon. Caused by the large width/thickness ratio, the stability of the steel tubes is low, and buckling appeared on each tube. Failure modes of the specimens are shown in Figure 5, and the compression force ($P$)-steel tube vertical strains relation curves are shown in Figure 6.

The failure processes of specimens are similar. At the beginning of load, the column deformation increased steadily with the increase of compression. According to the testing phenomenon and the counter bend point shown in Figure 4, buckling appeared on the tube surface, which
Figure 3: Specimen sizes.

Figure 4: Experimental devices.

Figure 5: Failure modes of the specimens: (a) Z650; (b) Z1200.
reached 30%–40% maximum bearing capacity without screws when loading under compression. When loading compression with screws reached 70%–80% maximum bearing capacity, buckling appeared on the tube surface. With increased compression deformation, buckling appeared more seriously. After loading reached the maximum bearing capacity, the corner of the steel tubes deformed seriously, the bearing capacity decreased quickly, and the steel tubes expand outwards. A tearing sound was made by adhesive tapes which are fixed wires, and some taps dropped because of the large tube deformation. Cracking could be heard from concrete in the tubes. It can be observed from the failure specimens after tests that buckling waves are all three-dimensional sinusoidal surface shaped. Vertical spans of the buckling waves with or without screws on the surfaces are all similar as the spacing distance of the screws.

2.4. P-Δ Curves. P-specimen deformation (Δ) curves are shown in Figure 7. It can be observed that bearing capacity of Z650 is 1032.00 kN, and the failure deformation is 0.89 mm; bearing capacity of Z1200 is 1052.90 kN, and failure deformation is 1.66 mm. According to the P-Δ curves, it can be observed that this type of column has low ductility. According to the failure process and P-Δ curves, it can be observed that the strength and stiffness of the corner areas and capacity to resist buckling of the tube plate are two important factors that influence the structural failure modes and bearing capacity. At the beginning of the test, the corner edges have higher stiffness than the steel tube plate, and the corner edges withstand greater pressure loads than the plate areas. These loads caused easy damage to the edges. In this period, edges and screws provide a constraining effect to limit plate buckling. When the load is nearly the bearing capacity, the concrete corners and steel corners are collapsed, and some screws are largely deformed. The constraining effect on the steel plate change becomes weaker, causing a rapid development of steel buckling. As a result of the large buckling deformation of steel tubes and less steel-provided ductility, the bearing capacity of the columns increased with a no-significant plastic stage and decreased suddenly, which is similar as plain concrete columns. Thus, improving the section size and bending stiffness of the steel corner areas is necessary.

3. Elastic Calculation of Tube Buckling

3.1. Interaction between Tubes, Concrete, and Screws. Each seam of the steel tube seams is connected by 2 team self-tapping screws. At the beginning of loading, screws and tube are not in complete close contact, gaps exist between the screw hats and the tube, and gaps are present between the screw bodies and holes on the tube. Since the screws cannot fully constrain the steel tubes at that time, tubes bulge out under vertical pressure until touching the screw hats. At the same time, tubes could also slide relative to the screws. The tube sliding and bulge phenomena are shown in Figure 8. After screws and tubes are fully touched, the screw hats restrained the tube buckling and lateral expansion. The expansion restraint caused a hoop effect of the steel tube on concrete. Furthermore, screws transfer vertical force between the concrete and steel tubes. This transfer effect can limit relative sliding between concrete and concrete, giving a full section the same compression displacement.

Interaction between concrete and the steel tube is shown in Figure 9. Buckling waves are all three-dimensional sinusoidal surface shaped. The buckling peak is farthest from the concrete, and the stress states of steel in buckling peaks are under vertical compression and lateral tension. Buckling troughs touched surfaces of the concrete, and these troughs are counter bend lines of the steel tube. In trough areas, the steel tube buckling toward the inside is limited, and the concrete and steel tubes compress to each other laterally. This compression effect makes a supporting to buckling
Figure 7: P-Δ curves.

Figure 8: Tube sliding and bugle phenomenon: (a) vertical sliding; (b) lateral expansion; (c) bulge out.

Figure 9: Interaction between concrete and steel.
steel, and it makes buckling still have the compression bearing capacity. Furthermore, compression between the steel and concrete contact surface and compression to the concrete, which is caused by the lateral tension of the steel tube, gives the concrete a triaxial compression stress state. Thus, the bearing capacity and column stiffness are not decreased immediately after steel buckling appearance.

3.2. Analysis of Small Deflection. According to research by Shi-Xu [19], the buckling steel area can be sampled as a thin plate model on which clamped restraint and concrete can be considered as a rigid base. According to small deflection theory and the principle of potential energy, the critical buckling load is as follows:

\[ P_{cr} = \frac{k_{cr} \pi^2 E}{12(1-\mu^2)} \left( \frac{t}{b} \right)^2, \]  \hspace{1cm} (1)

where Poisson’s ratio \( \mu = 0.3 \) and the steel plate wide-thickness ratio \( b/t = 200 \). For tube surfaces where there are no screws, buckling appeared when the mean pressure strain \( \varepsilon_p = 0.0005 \) and \( P_{cr} = 100 \text{ MPa} \); using these values into Eq. (1), it can be calculated that \( k_{cr} = 21.84 \). For tube surfaces where there are screws, buckling appeared when mean pressure strain \( \varepsilon_p = 0.0010 \) and \( P_{cr} = 200 \text{ MPa} \); using these values in Eq. (1), it can be calculated that \( k_{cr} = 43.68 \). Reflect screws not only provide steady seal connection to steel tubes but can also improve the buckling resistance ability of the tubes.

3.3. Large Deflection Analysis for Tube Elastic Buckling. The simplified calculation model for the buckling area is shown in Figure 10, where \( b \) is the lateral span of the buckling area (tube width) and \( h \) is the vertical span of the buckling area. Assuming that the concrete is a rigid substrate, ignoring the shearing effect in the steel plate and the bending resistance of the steel plate, the buckling area can be sampled as a steel film. Therefore, the boundary conditions of the buckling area are \( w(x = 0) = 0, w(x = b) = 0, (\partial w(x = 0)/\partial y) = 0, \) and \( (\partial w(x = b)/\partial y) = 0, \) and function of buckling is as follows:

\[ w(x, y) = \frac{a}{2} \cos \frac{\pi y}{h} \cos \frac{2\pi x}{b} - \frac{a}{2} \cos \frac{\pi y}{h} + \frac{a}{2} \cos \frac{2\pi x}{b} \]  \hspace{1cm} (2)

Strains of the buckling area are

\[ \varepsilon_x = \frac{\partial w}{\partial x} + \left( \frac{\partial w}{\partial x} \right)^2 = \frac{1}{2} \times \frac{a^2}{b^2} \pi^2 \sin^2 \frac{2\pi x}{b} \left( 1 - \cos \frac{\pi y}{h} \right)^2, \]

\[ \varepsilon_y = \frac{\partial w}{\partial y} \left( \frac{\partial w}{\partial y} \right)^2 = \frac{\partial w}{\partial y} + \frac{a^2}{2h^2} \pi^2 \sin \frac{\pi y}{h} \left( 1 - \cos \frac{2\pi x}{b} \right)^2. \]  \hspace{1cm} (3)

Lateral stress of the buckling area is

\[ \sigma_x = \frac{E}{1-\mu^2} (\varepsilon_x + \mu \varepsilon_y). \]  \hspace{1cm} (4)

The equivalent uniform stress is \( P_y = \int_0^b \sigma_y \, dx; \) \( P_y \) is balanced to \( p_h \) (components along the \( z \) axis of \( \sigma_x \), as shown in Figure 11). Thus, the equilibrium equation can be obtained:

\[ M_x P_y \times t \int_0^b \omega \left( y = \frac{h}{2} \right) dx = P_y \times t \times a \times b, \]  \hspace{1cm} (5)

\[ M_x = \int_0^b \int_0^{h/2} \sigma_x \cdot y \cdot \frac{\partial \omega}{\partial x} dx dy. \]  \hspace{1cm} (6)

These can be calculated according to Eqs. (5) and (6):

\[ P_y = \frac{1}{a \cdot b \cdot t} \int_0^b \int_0^{h/2} \left\{ E \left[ \frac{1}{1-\mu^2} \left( 1 - \cos \frac{2\pi x}{b} \right)^2 + \frac{1}{2} \mu \cdot a^2 \cdot 2h^2 \pi^2 \sin \frac{\pi y}{h} \left( 1 - \cos \frac{2\pi x}{b} \right)^2 \right] \right. \]

\[ \left. + \frac{1}{2} \mu \cdot a^2 \cdot 2h^2 \pi^2 \sin \frac{\pi y}{h} \left( 1 - \cos \frac{2\pi x}{b} \right)^2 + \frac{1}{2} \cdot \frac{\partial v}{\partial y} \cdot y \cdot \frac{a}{b} \pi \cdot \sin \frac{2\pi x}{b} \left( 1 - \cos \frac{\pi y}{h} \right) \right\} dx dy \]

\[ = \frac{E}{2(1-\mu^2)} \cdot \frac{1}{t} \cdot \frac{a^2}{\eta^4} J(\eta) + p_0, \]  \hspace{1cm} (7)

where

\[ J(\eta) = \int_0^b \int_0^{h/2} \left\{ \left[ \frac{h^2 \sin \frac{2\pi x}{b} \left( 1 - \cos \frac{\pi y}{h} \right)^2}{1 - \cos \frac{2\pi x}{b}} \right] \left[ \frac{\partial v}{\partial y} \cdot y \cdot \frac{a}{b} \pi \cdot \sin \frac{2\pi x}{b} \left( 1 - \cos \frac{\pi y}{h} \right) \right] dx dy, \] \hspace{1cm} (8)

\[ \eta = \frac{h}{b} \]

\[ p_0 = \frac{1}{2} \cdot \frac{1}{a \cdot b \cdot t} \int_0^b \int_0^{h/2} E \cdot \mu \cdot \frac{\partial v}{\partial y} \cdot \frac{\partial w}{\partial x} dx dy. \]

When \( w = 0, \) \( p_y = p_0 = P_{cr} \), so

\[ P_y = \frac{E}{2(1-\mu^2)} \cdot \frac{1}{t} \cdot \frac{a^2}{\eta^4} J(\eta) + P_{cr}. \]  \hspace{1cm} (9)

According to Eq. (9), after panel buckling, \( p_y \) increased with the increase of \( a^2 \), as shown in Figure 12. This means after buckling appeared, the equivalent uniform stress of the steel plate in the buckling area can reach yield stress, and it can supply enough designing stress.

4. Elastoplastic Analysis by FEM

4.1. Model Information. In this situation, the concrete damaged plasticity model (Lee and Fenves, 1998) of
ABAQUS is used. This concrete-damaged plasticity model includes the compressive strength enhancement of concrete resulting from confinement and the postpeak softening behavior resulting from concrete crush. According to "Code for design of concrete structures" (GB50010-2010), concrete compression stress-strain relationships are as follows:
\[ \sigma = (1 - d_c) E_c \varepsilon_c, \]
\[ d_c = \begin{cases} 1 & \text{if } x \leq 1, \\ 1 - \frac{\rho_c n}{\alpha_c (x-1)^2 + x} & \text{if } x > 1, \end{cases} \]
where \( f_c \) is the concrete strength and \( \varepsilon_c \) is the corresponding strain.

The yield criterion of steel is related to Mises yield strength:
\[ k = \frac{f_y}{\sqrt{3}}, \]
\[ k^2 = f_2 = \frac{1}{6} \left[ (\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2 \right], \]
where \( \sigma_1, \sigma_2, \) and \( \sigma_3 \) are principal stresses of steel and \( f_y \) is the steel yield strength.

The model sizes are same as that of the test specimens. The names of the models are 650 mm as C6 and 1200 mm as C1200.
As shown in Figure 13, the steel tube and concrete are solid elements, the self-tapping screws are simplified as truss elements, and the truss elements are the embedded region constrained to the entire model. The tangential contact model between the steel and concrete is Coulomb friction, and the normal contact model between steel and concrete is a hard contact model. The feet of the column model are fixed, and compression is applied on the top surface as deformation.

4.2. Results Analysis. $P-\Delta$ curves of the analysis results are shown and compared with the experimental results in Figure 14; the analysis results are highly fitted to the experimental results. As it is observed from the results, for model C650, $P_m = 1002.03$ kN, $\Delta_m = 0.89$ mm, $P_y = 973.71$ kN, $\Delta_y = 0.85$ mm, and $\Delta_u = 0.89$ mm; for model C1200, $P_m = 933.61$ kN, $\Delta_m = 1.64$ mm, $P_y = 967.44$ kN, $\Delta_y = 1.59$ mm, and $\Delta_u = 1.66$ mm, where $P_m$ is the maximum bearing capacity, $\Delta_m$ is the corresponding deformation; $P_y$ is the yield compression force, and $\Delta_y$ is the corresponding deformation; and $\Delta_u$ is the failure deformation. The deviation between the analysis results and the experimental results is no more than 10%.

The stress distributions of the concrete and steel tube models, when structural failures occur, are shown in Figure 15. Concrete corner edges in C650 and C1200 are crushed. Of these two models, in the vertical direction, concrete compression stress was gradually increasing from the bottom and top sides to the middle; in the lateral direction, the compression stress of the concrete corners is higher than that of the concrete middle parts. Upon C650 failure, most areas of the steel tube plate and all steel corner edges yield, the Misses stress of the corners are higher than that of steel tube plate; buckling appears on the middle part of tube, and the steel corner edge is collapsed. Upon C1200 failure, nearly all areas of the steel tube yield, and many buckling areas appear on the steel tube.

According to analysis results and the experimental results, the Misses stress of the corners is higher than that of the steel tube plate. It is a result of internal force distribution between steel tube plates and steel corner edges. These steel corner edges have higher bending rigidity than steel tube plates and can keep stable when the steel tube plate buckling occurs. Thus, from the top side and the bottom side to the middle part of the tube, compression stress of the tube plates will transfer to the steel corner edges in a shear stress form. This internal force distribution is shown in Figure 16. This phenomenon makes the steel corner edges to collapse easily, as shown in Figure 17.

5. Column Section Optimization

To address the problem that steel corner edges are easily collapsed, an optimization model L650 is developed, and its
steel tube is shown in Figure 18. In L650, corner edges are strengthened to be L-shaped steels and are connected to 1 mm thickness steel plates using self-tapping screws. This optimization section is named the “L-typed section.” According to the analysis results, the deformation and stress of the L650 tube when column failure occurs are presented in Figure 19. It can be observed that many buckling areas appeared on the steel tube, and the steel plates stress is very low. Only strengthened steel corners but no additional restraint for the steel plate will make corner areas bear more of the internal force. Thus, the steel corners are easier to damage, and plate buckling can be more serious. It is necessary to increase bending stiffness of steel plates and make more constraint on the plates. This new method should not increase steel consumption greatly and must use an easy construction method.

Aimed at improving stability of tube plates, an optimization section which is named as “B-typed section” is proposed and shown in Figure 20. In this new type section, tube plates are changed to be closed-type steel plates. In the B-typed section, the ridge of the closed-type steel plate is implanted in concrete, and the ridges increased the inertia moment of the steel plate; these increased the stability of the sample. Models B650 and B1200 are in the B-typed section, and their lengths are 650 mm and 1200 mm, respectively. Compression of these two models was modeled in software Abaqus, and the concrete stress and steel stress of B650 and B1200 when model failure occurred are shown in Figure 21. Compared with L650, the steel tube stress in the B-typed section models is more uniform, both in steel corner edges and tube plates yield, and buckling of the steel tube is controlled. Compared with C650 and C1200, more concrete...
areas are crushed in the B-typed section models, which mean concrete stresses are more fully used.

$P-\Delta$ curves of these models are shown in Figure 22, and a factor $\mu = \Delta_y/\Delta_u$ reflects ductility. Compared with B650, C650 $P_m$ increases 30.12%, failure deformation $\Delta_u$ increases 29.12%, and $\mu$ increases 8.08%. Compared with L650, C650 $P_m$ increases 56.77%, failure deformation $\Delta_u$ increases 26.07%, and $\mu$ increases 3.89%. Compared with B1200, C1200 $P_m$ increases 11.39%, failure deformation $\Delta_u$ increases 3.01%, and $\mu$ increases 2.37%. The out-of-plane relative deformation curves are drawn in Figure 23. Of the 650 mm length column models, L650 is the most seriously deformed out-of-plane, and B650 is the least deformed. B1200 is less deformed out-of-plane than C1200. These results reflect that the B-type section can address both problems of corner collapse and steel plate buckling. Thus, this new type section can make better use of the concrete material and steel material stress and increase the compression bearing capacity of the column.

6. Conclusion

Experimental and analysis results prove that self-tapping screws can connect tube seams of square thin-walled CFST columns firmly. Moreover, failure modes of square thin-walled CFST columns are steel tube plate buckling and then the collapse of steel and concrete in some corner edge areas. Causes of these failure modes including the following: (1) the steel tube is too thin to maintain stability; (2) the internal force distribution between the steel tube plates and
the steel corner edges make steel corner edges easier to collapse.

Interaction between concrete and steel supports buckling steel and creates a triaxial compression stress state in concrete. This causes the bearing capacity and column stiffness not to decrease immediately after the appearance of steel buckling. Furthermore, according to Eq. (9), after panel buckling, $p_y$ is the two increasing function of the deflection $2a$, and steel can supply enough designing stress. However, collapse of steel and concrete in the corner edge areas caused the column bearing capacity to decrease quickly.

Aiming at failure modes of square thin-walled CFST columns, a B-type section is proposed as an improvement
scheme. Comparing the analysis results, the B-type section can address both the problems of corner collapse and steel plate buckling. Thus, this new type section can better make full use of the stress of the concrete material and steel material; this type section can also increase the compression bearing capacity of the column.

Compared with a traditional thin-walled CFST column section (C-type), the proposed method has the following improvements:

1. Section area and stiffness of steel corners is increased, and the proposed method provided stronger bearing capacity and higher stability. The method makes steel corners be broken later than concrete collapse, providing both higher bearing capacity and better ductility in B-type columns.

2. Larger bending stiffness of closed-type steel plates and embedding effect between plate and concrete limit buckling of the steel plate are exhibited. As a result, steel plates provide more bearing capacity, and the material is more fully used.

3. In a B-type section, the collapse risk of steel corners is decreased, and the column ductility is increased because more steel plates have limited buckling and can join vertical plastic deformation.

Figure 23: Out-of-plane relative deformation curves, where $H$ is the column length: (a) 650 mm column models; (b) 1200 mm column models.
Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that there are no conflicts of interest.

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