Research Article

Study on Axial Compressive Capacity of FRP-Confined Concrete-Filled Steel Tubes and Its Comparisons with Other Composite Structural Systems

Jun Deng, Yifeng Zheng, Yi Wang, Tonghua Liu, and Hui Li

School of Civil and Transportation Engineering, Guangdong University of Technology, Guangzhou 510006, China

Correspondence should be addressed to Jun Deng; jdeng@gdut.edu.cn

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Concrete-filled steel tubular (CFST) columns have been widely used for constructions in recent decades because of their high axial strength. In CFSTs, however, steel tubes are susceptible to degradation due to corrosion, which results in the decrease of axial strength of CFSTs. To further improve the axial strength of CFST columns, carbon fiber reinforced polymer (CFRP) sheets and basalt fiber reinforced polymer (BFRP) sheets are applied to wrap the CFSTs. This paper presents an experimental study on the axial compressive capacity of CFRP-confined CFSTs and BFRP-confined CFSTs, which verified the analytical model with considering the effect of concrete self-stressing. CFSTs wrapped with FRP exhibited a higher ductile behavior. Wrapping with CFRP and BFRP improves the axial compressive capacity of CFSTs by 61.4% and 17.7%, respectively. Compared with the previous composite structural systems of concrete-filled FRP tubes (CFFT) and double-skin tubular columns (DSTCs), FRP-confined CFSTs were convenient in reinforcing existing structures because of softness of the FRP sheets. Moreover, axial compressive capacity of CFSTs wrapped with CFRP sheets was higher than CFFT and DSTCs, while the compressive strength of DSTCs was higher than the retrofitted CFSTs.

1. Introduction

In a conventional concrete-filled steel tubular (CFST) column system, concrete was filled in steel tubes, which typically continued throughout several stories or the full-height of a building [1, 2]. After decades of development, CFST columns have been widely used in many buildings.

In CFSTs, however, steel tubes are susceptible to degradation due to corrosion, which results in the reducing of axial strength of CFSTs. To overcome the disadvantages, Xiao [3] proposed the FRP strengthened CFST (FCFST) columns. Both the inward and the outward buckling deformations of the steel tube in the strengthened CFSTs were constrained, thereby substantially enhancing the ductility and strength of the column. Thereafter, many studies were conducted by Xiao et al. [4–6] and other researchers [5, 7–11] on the effectiveness of FRP wrapping in improving the structural behavior of CFST columns. Teng et al. [12] developed a stress-strain model for confined concrete in FCFSTs.

Xue et al. [13] have found that shrinkage of the filled concrete can cause separation between steel tube and concrete, and Ghafari et al. [14] have found that shrinkage can reduce the durability of the structures. Expansive cements, which can increase the volume of concrete due to chemical reactions, can reduce the shrinkage of concrete. Therefore, the self-stressing concrete-filled steel tubes (SSCFSTs) were proposed as they have higher axial strength than CFSTs. A number of experimental studies [15, 16] on SSCFST have been conducted to understand its mechanical performance, and design theory [17] was proposed accordingly. Some models have been developed to predict the load capacity of the FCFST columns [18, 19]; however, the effect of the expansion of the concrete was not considered.

In addition, CFST is not the only system of concrete-filled tubular columns. Ozbakkaloglu et al. [20–22] conducted studies on the axial compressive behavior of concrete-filled FRP tubes (CFFT) and double-skin tubular columns (DSTCs). CFSTs were made of steel tubes, while CFFT were

FIGURE 1: Stress distribution in SSCFST.

made of FRP tubes. DSTCs were made of both steel tubes and FRP tubes, including hollow core (DSTC-H) and core-filled (DSTC-F). Since the characteristics and application limitations of each system are still not clear, the comparisons of these four systems are needed.

This paper gives a theoretical model for obtaining the axial compressive capacity of FCFSTs, in which the effect of self-stressing was considered. Besides, experimental study was also conducted to investigate the behavior of CFSTs wrapped with CFRP sheets (CCFSTs) and BFRP sheets (BCFSTs) subjected to axial compression. Finally, our test results were compared with existing test results of CFFT, DSTC-H, and DSTC-F.

2. Analytical Study

2.1. Basic Assumptions. The following assumptions are made in this analytical study:

- (1) The interfaces between the concrete and the steel tube, the steel tube and the FRP sheet are constrained.
- (2) The radial stress in the steel tube is ignored and the steel tube is under biaxial stress.
- (3) The FRP material is linear elastic.
- (4) Only the hoop stress in the FRP sheet is considered; that is, the stress along the fiber direction is considered and the radial stress and the longitudinal stress are ignored.

2.2. Axial Compressive Capacity. Since the concrete in the steel tube is under triaxial compression, the axial compressive strength of concrete σ_{cl} which considered both the self-stressing and axial compression can be expressed as [23]

$$\sigma_{cl} = f_{ck} + k(\sigma_{cr} + \sigma_{ss}), \quad (1)$$

where f_{ck} is the standard compressive strength of the concrete, σ_r the lateral pressure of the concrete, σ_{cr} the lateral stress of the concrete core caused by axial compression, σ_{ss} the

initial lateral stress caused by expansion of the self-stressing concrete, and k the lateral stress coefficient. k is normally between 3 and 5 in accordance with the experimental results [23] and assumed to be 4 in this study.

The critical state of the steel tube is analyzed using the maximum shear stress theory, known as Tresca yield criterion. It can be expressed as

$$\sigma_{st} - \sigma_{sl} + \sigma'_{st} = f_y, \quad (2)$$

where σ_{st} and σ_{sl} are the hoop tensile stress and axial compressive stress of the steel tube, respectively. σ'_{st} is the hoop tensile stress of steel tube that caused by self-stressing and f_y is the yield strength of the steel tube.

According to the basic assumption (1) and the stress-strain relationships of FRP and steel tube, the hoop stress of the FRP σ_{ft} can be expressed as

$$\sigma_{ft} = \frac{E_f}{E_s} [(1 - \mu_s) \sigma_{st} - \mu_s \sigma'_{st} + \mu_s f_y], \quad (3)$$

where E_s , μ_s , and E_f are Poisson's ratio and Young's modulus of the steel tube and the Young's modulus of the FRP sheet, respectively.

Figure 1 shows the stress distribution in SSCFST under axial compression. According to the stress balances in Figure 1, σ_{cr} and σ_{ss} can be expressed as

$$\sigma_{cr} = \sigma_{ft} \frac{A_f}{2A_c} + \sigma_{st} \frac{A_s}{2A_c}, \quad (4)$$

$$\sigma_{ss} = \frac{A_s}{2A_c} \sigma'_{st}, \quad (5)$$

where A_c , A_s , and A_f are the cross-sectional areas of the concrete, the steel tube, and the FRP, respectively.

The axial compressive capacity of the FRP-SSCFST can be described as

$$N_u = A_c \sigma_{cl} + A_s \sigma_{sl}. \quad (6)$$

FIGURE 2: Axial compressive capacity with the number of FRP layers.

This study considers the following failure modes of the FRP-SSCFST: steel tube buckling and FRP sheet rupturing.

(1) When the failure mode is steel tube buckling, the hoop tensile stress of the steel tube σ_{st} is the yield stress f_y . Substituting (1), (2), (3), (4), and (5) into (6) gives axial compressive capacity as

$$N_u = A_c f_{ck} \left[1 + 2\xi_s + 2\frac{E_f}{E_s} \xi_f + \left(6 - \frac{4E_f A_f}{E_s A_s} \mu_s \right) \frac{\sigma_{ss}}{f_{ck}} \right], \quad (7)$$

where

$$\xi_s = \frac{A_s f_y}{A_c f_{ck}}, \quad (8)$$

$$\xi_f = \frac{A_f f_f}{A_c f_{ck}}.$$

ξ_s and ξ_f are defined as the coefficients of the constraint effect caused by the steel tube and the FRP sheet, respectively.

(2) When the failure mode is FRP sheet rupturing, the hoop tensile stress of the steel tube σ_{st} is the yield stress f_y and the hoop tensile stress of the FRP σ_{ft} is the ultimate stress f_f . Substituting (1), (2), and (4) into (6) gives axial compressive capacity as

$$N_u = A_c f_{ck} \left(1 + 2\xi_s + 2\xi_f + 6 \frac{\sigma_{ss}}{f_{ck}} \right). \quad (9)$$

2.3. Parametric Analysis. Parametric analysis was performed to investigate the effects of the number of FRP layers and the self-stressing on the axial compressive capacity. The less one between the results calculated by (7) and (9) was the axial compressive capacity of the retrofitted CFSTs.

Figure 2 plots the axial compressive capacity versus the number of layers of FRP sheets. It shows that the axial compressive capacity increased with the number of FRP

FIGURE 3: Axial compressive capacity with the self-stress.

TABLE 1: Material parameters of CFST.

Materials	Yield strength (MPa)	28-day compressive strength (MPa)	Young's modulus (GPa)
steel	275	—	201
concrete	—	31.2	3.35×10^{-5}

Note. The mix proportions of the concrete were 190:400:786:1065:8 (water : cement : sand : aggregate : water reducer).

TABLE 2: Material parameters of FRP.

FRP types	Slice thickness (mm)	Tensile strength (MPa)	Young's modulus (GPa)
CFRP	0.167	2878	244
BFRP	0.167	1526	71

layers and the axial strength of CCFSTs is higher than that of BCFSTs. This is because the lateral restraint can be effectively enhanced with the thickness and ultimate tensile strength of FRP, thereby increasing the axial compressive capacity.

Figure 3 plots the axial compressive capacity versus self-stressing level of the filled concrete. It shows that the axial compressive capacity increased with the self-stress. This is because that the lateral restraint increased with the self-stress, thereby increasing the axial compressive capacity.

3. Experimental Study

In total, 11 specimens were prepared, including 3 CFSTs, 4 CCFSTs, and 4 BCFSTs.

Table 1 shows the parameters of steel and concrete of CFSTs. Table 2 shows the parameters of CFRP and BFRP.

After cutting the steel tubes into the desired dimensions, specimens were cast. Then two layers of FRP were wrapped in the hoop direction covering the whole steel tube after 28 days curing.

The tests were carried out in a servohydraulic test machine with a maximum capacity of 10000 kN. The load was

FIGURE 4: Strain gauges arrangement for specimens.

FIGURE 5: Load-displacement curves.

applied at a constant rate of 0.5 kN/s during the elastic stage and at a constant rate of 0.25 mm/min during the plastic stage until the specimens reached its ultimate axial compressive capacity. Four pairs of 3 mm long strain gauges were mounted on both the exterior of the steel tubes and FRP at the mid-height of the specimens to capture the longitudinal and hoop strains. All data was automatically recorded by a data logging system (TDS-530). The strain gauges' arrangement was shown in Figure 4.

4. Experimental Results and Discussions

4.1. Strength and Stiffness of the Specimens. The load versus displacement of the specimens is shown in Figure 5. All specimens have the same loading steps, which can be classified into four stages: elastic stage, elastic-plastic stage, plastic stage, and failure stage. During the elastic stage, the load is apportioned according to the proportion of stiffness between FRP, steel, and concrete. Therefore, after calculation, it can be seen that the load steel suffered is more than 80% of the whole load, which is much greater than FRP and concrete. The stiffness of steel plays a leading role in this stage, which results in the stiffness of FCFST specimens closed to that of the CFST

FIGURE 6: Load-strain curves of CCFST-3.

specimens. During the elastic-plastic stage, the stiffness of the strengthened specimens, especially the CCFST specimens, was significantly higher than the CFST specimens. Because outward buckling deformation of the steel tube was constrained by FRP wrapping, the CFRP has more effective confinement than BFRP because of its higher Young's modulus.

The average axial compressive capacity of CFSTs, CCFSTs, and BCFSTs was 1207 kN, 1949 kN and 1421 kN, respectively. The CFRP and BFRP strengthening enhanced the axial compressive capacity of CFSTs by 61.4% and 17.7%, respectively. The average ultimate displacement of CFSTs, CCFSTs, and BCFSTs was 16 mm, 30 mm, and 20 mm, respectively. Besides, CFRP and BFRP strengthening enhanced the ductility by 87.5% and 25%, respectively. It indicates that the strength and ductility of the CFSTs can be significantly improved by FRP wrapping, especially CFRP wrapping.

4.2. Strains of the Specimens. The hoop strain of CFRP and hoop and longitudinal strain of steel tube recorded during the tests of CCFST-3 and BCFST-1 are shown in Figures 6 and 7, respectively. It can be seen that the hoop strains of both CFRP and steel tube were very small during the elastic stage. This is because that the steel tube and the concrete carried on the axial load and the confinement of the steel tube and FRP sheets was not pronounced. The longitudinal strains of the steel tubes developed as elastic-perfectly plastic body. Comparing Figures 6 and 7, the ultimate longitudinal strain of the steel tubes in the CCFST is larger than that in the BCFST, which shows that the confinement of CFRP wrapping is more effective. The load-strain curves of the FRP developed consistently with those of the steel tubes, which indicates that the FRP wraps and steel tube can work together well. Similar results were obtained from the other strengthened specimens.

4.3. Failure Modes. All specimens were loaded to failure to understand the effect of the FRP on the axial strength of CFSTs. The failure modes of typical specimens were shown

FIGURE 7: Load-strain curves of BCFST-1.

TABLE 3: Comparisons of axial compressive capacity from experiment and analytical results.

Specimens	Axial compressive capacity (kN)		Deviations (%)
	Average experimental results	Analytical results	
CFSTs	1207.3	1198	-4.01
CCFSTs	1948.8	2037	4.53
BCFSTs	1420.5	1247	-12.21

in Figure 8. As shown in Figure 8(a), the outward buckling of CFSTs was observed in the middle of the steel tube at failure load. In the case of FRP strengthened specimens, snapping sound of fiber was observed after steel tube yielded. The reason could be that FRP wrapping provided significant restraint against the lateral deformations of concrete core and steel tube and the FRP was subjected to tension in the hoop direction. When the FRP reached its ultimate strain, rupture of FRP occurred followed by the outward buckling of steel tube in the middle of the specimens, as shown in Figures 8(b) and 8(c).

4.4. Validating Analytical Analysis. The axial compressive capacity of the specimens obtained from tests was compared to the corresponding analytical results calculated by (7) or (9), as presented in Table 3. The agreement between the experimental and analytical results of CFSTs was reached. The deviations of experimental and analytical results of CCFSTs and BCFSTs are 4.53% and -12.21%, respectively, which is considered acceptable.

5. Comparisons of CFST, CFFT, DSTC-H, and DSTC-F

Over the past five years, significant numbers of studies have been reported on the composite columns such as CFFT, DSTC, and FCFST, in which Ozbakkaloglu et al. [20–22]

(a) CFST

(b) CCFST

(c) BCFST

FIGURE 8: Failure modes of specimens.

conducted studies on axial compressive behavior of CFFT, DSTC-H, and DSTC-F made of normal-strength and high-strength concrete, making great contribution to the new composite structure systems. To date, there are few literatures on the contrasted analysis of axial compressive strength and capacity of the different composite columns. The schematics of the four types of columns in literatures [20, 21] and this study were shown in Figure 9.

Table 4 shows the axial compressive capacity (N_u) and axial compressive strength (f_{cc}) obtained from the experimental results of the four types of columns made of CFRP. It demonstrates that the axial compressive capacity (N_u) of

FIGURE 9: The schematics of four columns.

TABLE 4: Comparisons of CCFST, CFFT, DSTC-H, and DSTC-F.

Type	Specimen	t_f (mm)	t_s (mm)	f_c (MPa)	f_y (MPa)	f_t (MPa)	N_u (kN)	f_{cc} (MPa)
CCFST	average	0.334	2.0	31.2	275	2878	1948.8	91.2
CFFT [20]	N-150C2L-2	0.234	—	34.6	—	3626	1104.5	60.9
DSTC-H [21]	DSTC-2	0.234	3.2	37.0	302	3626	955.0	99.9
DSTC-F [21]	DSTC-4	0.234	3.2	36.7	302	3626	1322.0	74.9

Note. f_c = the 28-day compressive strength of concrete; t_f = the total nominal thickness of CFRP; t_s = the thickness of steel tube; CFRP tubes of CFFT, DSTC-H, and DSTC-F were made of two layers of CFRP (nominal thickness is 0.117 mm/ply), while CCFSTs were wrapped with two layers of CFRP sheets (nominal thickness is 0.167 mm/ply).

CCFSTs is higher than those of CFFT, DSTC-H, and DSTC-F. However, the cross-sectional area of CCFSTs is the largest in the four types of columns. The axial compressive strength (f_{cc}) of DSTC-H is greater than CCFSTs and CFFT. Overall, the four types of columns have their own advantages: (a) CFFTs do not need steel and DSTC-H can cost less concrete; (b) DSTC-H is lighter than CCFST and CFFT; (c) CFFT, DSTC-H, and DSTC-F are used as new structures, while CCFST is generally applied to reinforce existing structures; (d) as the novel structures, CFFT and DSTC are applied by well-shaped columns in present; (e) CCFST is conveniently utilized in the structures of the steel tube columns reinforced in active service, which benefits from softness of the FRP sheets.

6. Conclusions

This paper investigated the axial compressive capacity and strength of FCFSTs. Theoretical study was presented to predict the nominal axial compressive strength of FCFSTs, in which the effect of self-stressing concrete was considered. In addition, the experimental study was also conducted to verify the model and investigate the behavior of CCFSTs and BCFSTs subjected to axial compression. As discussed, the following conclusions can be reached.

(1) The axial compressive capacity of the CFSTs wrapped with CFRP and BFRP was improved by 61.4% and 17.7%, respectively. Besides, the ductility of the CFSTs was improved as well.

(2) The good agreement between the experimental axial strength and the analytical results confirmed the validity of the theoretical model.

(3) CCFST can be conveniently and effectively utilized in the existing structure strengthening, while CFFT, DSTC-H, and DSTC-F are applied by well-shaped column as novel structures. Among the four types of columns, DSTC-H has the highest compressive strength.

Competing Interests

The authors declare that there is no conflict of interests regarding the publication of this paper.

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