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Engineering Structures 26 (2004) 907–915
ENGINEERING STRUCTURES
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Experimental study on the strength and ductility of steel tubular columns filled with steel-reinforced concrete

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Received 1 November 2003; received in revised form 28 January 2004; accepted 16 February 2004

Abstract

The steel tubular column filled with steel-reinforced concrete is formed by inserting a steel section into a concrete-filled steel tube. The objective of this study is to investigate the strength and ductility of composite columns subjected to axial compressive loads. For this purpose, an experimental study was conducted for 16 composite columns. The main test parameters were the concrete strength, the confinement index (θ), the structural steel index (ρ) and the length-to-diameter ratio. From test results, the strength and deformation characteristics of these composite columns were reported. The results showed that composite columns had very high strength, ductility and capacity of energy absorption because of the interaction between the steel tube, steel section and concrete. Formulas for the prediction of the ultimate strength of the composite column were also proposed in this paper, and calculated results are in good agreement with test results.
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Keywords: Composite column; Ductility; Ultimate strength

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1. Introduction

Composite steel-concrete columns have been widely used in high-rise and long-span buildings. The main advantages of composite columns are high load-carrying capacity, inherent ductility and toughness. For a given load, the composite column has a smaller cross-section and higher strength-to-weight ratio than conventional reinforced concrete member. Thus, the use of composite columns in buildings will lead to significant savings in material and increase in net floor space.

There are two main types of composite columns, namely, the concrete-encased steel column and the concrete-filled steel tubular (CFST) column. In the case of concrete-encased steel columns, the steel core can improve the shear resistance of the column. The concrete encasement makes the steel core more effective against both local and overall buckling [1]. The concrete encasement also provides protection against corrosion from chemical and insulation from rapid temperature rise in the presence of fire. The main disadvantage of concrete-encased steel columns is that a reinforced cage is required in order to prevent the concrete cover from spalling at low load levels. Furthermore, concrete-encased steel columns require extensive formwork during construction, especially at the beam–column connections. On the other hand, the CFST column requires no reinforced cage and no formwork. Concrete inside the steel tube contributes to stability of the steel tube, and the steel tube in turn provides effective lateral confinement to the concrete. This type of continuous confinement can improve the ductility and strength of normal and high-strength concrete [2,3]. The main disadvantage of the CFST column is that the steel tube is exposed, thus leading to a lower fire resistance compared with concrete-encased steel columns.

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doi:10.1016/j.engstruct.2004.02.009

Nomenclature

A_t, A_s, A_c	area of cross-section of steel tube, steel section and concrete, respectively
E_t, E_s, E_c	elastic modulus of steel tube, steel section and concrete, respectively
I_t, I_s, I_c	second moment of area of steel tube, steel section and concrete, respectively
f_{cu}	cube strength of concrete
f_c	compression strength of the unconfined concrete column
f_y^t	uniaxial yield stress of steel tube
f_y^s	uniaxial yield stress of steel section
N	compression force on column
N_0	ultimate strength of the stub column under axial compression
N_u	ultimate strength of the column under axial compression
φ	reduction factor due to the slenderness of column
D	outside diameter of steel tube
t	thickness of steel tube
L	effective length of column
ρ	structural steel index, $\rho = A_s f_y^s/(A_c f_c)$
θ	confinement index, $\theta = A_t f_y^t/(A_c f_c)$
$\bar{\lambda}$	relative slenderness of column

On the basis of studies of concrete-encased steel columns and CFST columns, other types of composite columns have been developed in recent years. Chicoine et al. [4] studied the behaviour and strength of partially encased composite columns with built-up shapes. Elchalakani et al. [5] studied the load-carrying capacity of concrete-filled double-skin composite columns. Aboutaha and Machado [6] studied the load-carrying capacity of steel tubular columns filled with reinforced concrete. These studies indicated that the strength and ductility of concrete-encased steel columns or CFST columns could be further enhanced by their proposed methods. Apart from using the various types of insulating methods, the CFST column has been studied for some time to determine how to enhance its fire resistance characteristics without external fire protection. Lie and Kodur [7] studied the fire resistance of steel tubular columns filled with reinforced concrete. Kodur and Lie [8] studied the fire resistance of steel tubular columns filled with fibre-reinforced concrete. These studies showed that the fire resistance of columns could be improved when reinforced concrete or fibre-reinforced concrete was filled in steel tubes. In reinforced CFST columns, the presence of bars also contributed to the strength of composite columns. Kodur [9] summarised their studies and pointed out that for slender columns and for columns with eccentric loads or very high axial compression, steel tubular columns should be filled with reinforced concrete instead of plain concrete to enhance the strength and the fire resistance of columns.

Based on the above research, the composite column formed by inserting steel sections into concrete-filled steel tubes should have the advantages of both concrete-encased steel columns and CFST columns. The cross-section of composite columns is shown in Fig. 1, and composite columns will be called steel tubular columns filled with steel-reinforced concrete hereafter. The main objective of this study is to investigate the strength and ductility of this type of composite column subjected to axial compressive loads. For this purpose, 16 composite columns were tested. The concrete strength (f_c), the confinement index ($\theta = A_t f_y^t/(A_c f_c)$), the structural steel index ($\rho = A_s f_y^s/(A_c f_c)$) and the length-to-diameter (L/D) ratio were the primary parameters in the test programme. These parameters were studied to determine their influence on the strength and ductility of the composite columns.

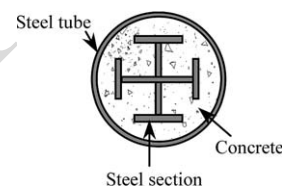

Fig. 1. Cross-section of the composite column.

2. Test method

2.1. Characteristics of specimens

A total of 16 specimens were constructed and tested under axial compressive loads. Of these 16 specimens, 15 were steel tubular columns filled with steel-reinforced concrete columns, and one was a CFST

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Table 1
Mix proportions of concrete

Mix type	Cement (kg/m^3)	Sand (kg/m^3)	Coarse aggregate (kg/m^3)	Water (kg/m^3)	Superplasticizer (kg/m^3)	Water/cement ratio
HSC	470	695	1053	179	14.1	0.38
NC	360	676	1177	197	–	0.547

column. Two types of concrete, normal concrete (NC) and high-strength concrete (HSC), three types of steel tube—diameter-to-thickness ratios between 45.4 and 72, two types of steel section—100 and 120 mm height of cross-section, combined with three L/D ratios—2.8, 7 and 10 were used to study their influences on the behaviour of the composite columns.

The concrete mix proportions are given in Table 1. In accordance with relevant Chinese standards, when the specimens were cast, a large number of concrete cubes and prisms were made to establish the compression strength of unconfined concrete column (f_c) and modulus of elasticity. The average compressive strength of the cubes at 28 days was 36.8 MPa for NC and 65.6 MPa for HSC, respectively. The average cube strength at the time of test was 44.2 MPa for NC and 68.9 MPa for HSC, respectively. The modulus of elasticity was 33 490 MPa for NC and 42 480 MPa for HSC, respectively. The value of f_c is determined as 67% of the cube strength.

The details of test variables are given in Table 2.

2.2. Specimen preparation

In manufacturing the steel framework, the steel tubes and steel sections were accurately cut to size firstly. Then a square steel plate was welded to each steel section ends, with special attention paid to the centering and perpendicularity of the steel plate. The steel section was then coated with a steel tube. The steel tube was adjusted carefully in order to assure the steel section lies in the core of the steel tube. Finally, the steel tube was also welded to the steel plate. Fig. 2 shows the welded steel framework.

The specimens were cast on a low frequency vibrating table to allow compaction of the concrete. Then the specimens were covered using polyurethane sheets and stored at ambient temperature in the concrete laboratory until testing. A very small amount of longitudinal shrinkage occurred at the top of specimens during curing. So prior to testing, the top surface of each specimen was roughened with a wire brush and a thin layer of cement was poured on the rough surface to fill the longitudinal gap. This ensured loading on the concrete, the steel section and the steel tube simultaneously.

2.3. Test setup and procedures

All specimens were tested in a 5000 kN capacity universal testing machine. The test setup for the specimens is shown in Fig. 3. Eight strain gauges were glued to the flanges and webs of each steel section at mid-height to measure the axial and lateral strains. Another eight strain gauges were placed on the external surface of the steel tube at mid-height to measure axial deformations and perimeter expansion of the steel tube wall.

Before testing, the 45 mm thick clamping plates were bolted to the top and bottom of the specimen to enhance the strength at the ends. Two linear variable differential transducers (LVDTs) were placed between the top and bottom clamping plates to monitor the axial deformation. Three LVDTs were also used to measure the lateral deflection at mid-height and quarter-height of specimens HM-1, HM-2, HL-1 and HL-2. A computerised data-acquisition system was used to collect the load, deformation and strain data.

Two test approaches were used in this study. In the first approach, the specimens were loaded under monotonically increasing axial compressive loads. In the second approach, the specimens were tested under cyclic axial compressive loads, namely unloading and reloading. Column loads were applied at a very slow rate so that the local buckling behaviour of the composite columns could be observed. The tests were terminated when considerable deformation occurred and local wall buckling was observed.

3. Results and discussions

3.1. Load-deformation relationship and failure mode

The ultimate strength of each specimen is given in Table 2. The test load versus axial strain curves of stub specimens ($L/D = 2.8$) are presented in Fig. 4, while the typical failure modes of stub specimens are shown in Fig. 5. The compressive behaviour of stub specimens can basically be described as follows. Generally, the load–strain curve was close to linearity before the load was approximately 60% of the ultimate strength N_u for NC specimens and 70% of N_u for HSC specimens. Beyond this load level, the steel began to yield and the load–strain curve diverged significantly from its initial linearity. Because the core concrete was confined by the

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Table 2
Properties of specimens

Specimen no.	L (mm)	D (mm)	t (mm)	D/t ratio	L/D ratio	f_y^t (MPa)	E_t (GPa)	A_s (mm^2)	f_y^s (MPa)	E_s (GPa)	I_s (cm^4)	f_c (MPa)	θ	ρ	N_u (kN)	Loading method
NS-A1	465	166	2.7	61.5	2.8	318	191	2324	288	207	208.6	29.6	0.830	1.261	2350	Monotonic
NS-A2	465	166	2.7	61.5	2.8	318	191	2324	288	207	208.6	29.6	0.830	1.261	2100	Monotonic
NS-B1	470	168	3.7	45.4	2.8	318	191	2324	288	207	208.6	29.6	1.144	1.261	2640	Monotonic
HS-A1	465	166	2.7	61.5	2.8	318	191	2324	288	207	208.6	46.2	0.532	0.808	2700	Monotonic
HS-A2	465	166	2.7	61.5	2.8	318	191	2324	288	207	208.6	46.2	0.532	0.808	2650	Cyclic
HS-B1	470	168	3.7	45.4	2.8	318	191	2324	288	207	208.6	46.2	0.733	0.808	2835	Monotonic
HS-B2	470	168	3.7	45.4	2.8	318	191	2324	288	207	208.6	46.2	0.733	0.808	2862	Cyclic
HS-C1	605	216	3.0	72.0	2.8	269	190	2324	288	207	208.6	46.2	0.362	0.448	3640	Monotonic
HS-C2	605	216	3.0	72.0	2.8	269	190	2324	288	207	208.6	46.2	0.362	0.448	3550	Cyclic
HS-D1	605	216	3.0	72.0	2.8	269	190	3570	314	197	500.1	46.2	0.376	0.781	4130	Monotonic
HS-D2	605	216	3.0	72.0	2.8	269	190	3570	314	197	500.1	46.2	0.376	0.781	4330	Cyclic
HS-E1	470	168	3.7	45.4	2.8	318	191	–	–	–	–	46.2	0.649	–	2358	Monotonic
HM-1	1176	168	3.7	45.4	7.0	318	191	2324	288	207	208.6	46.2	0.733	0.808	2830	Monotonic
HM-2	1176	168	3.7	45.4	7.0	318	191	2324	288	207	208.6	46.2	0.733	0.808	2735	Monotonic
HL-1	1680	168	3.7	45.4	10.0	318	191	2324	288	207	208.6	46.2	0.733	0.808	2468	Monotonic
HL-2	1680	168	3.7	45.4	10.0	318	191	2324	288	207	208.6	46.2	0.733	0.808	2430	Monotonic

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Fig. 2. Welded steel framework.

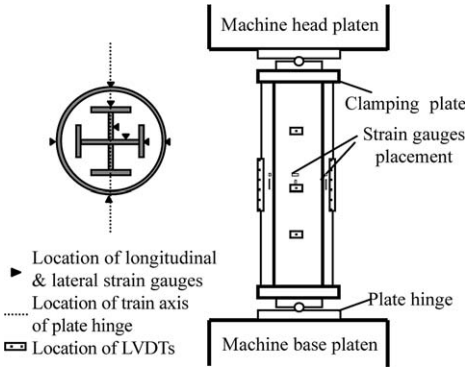

Fig. 3. Schematic of test specimen and test setup.

steel tube, the ultimate strength of the composite column would be considerably greater than the sum of the uncoupled steel and concrete failure loads. The post-yield behaviour of each specimen was affected by f_c, θ and ρ. Once N_u was reached, a significant decrease in

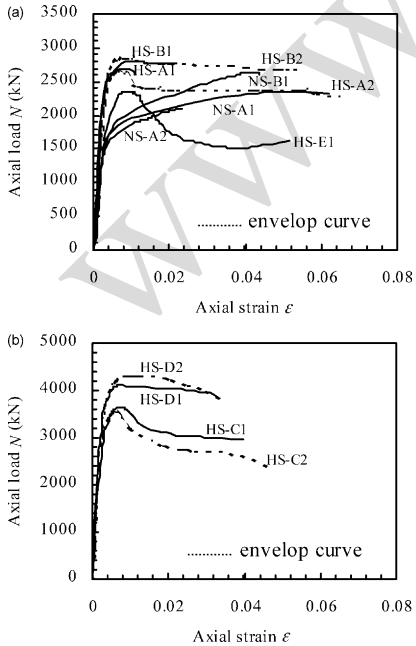

Fig. 4. Load–strain curves of stub specimens ($L/D = 2.8$).

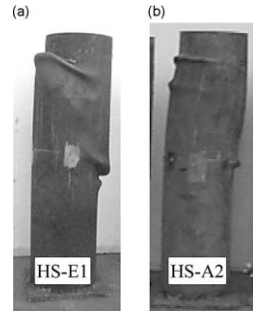

Fig. 5. Typical failure patterns.

the axial load occurred for the CFST specimens. At the same time, two bulges occurred on the surface of the steel tube (see Fig. 5(a)). There obviously existed a shear failure plane between the two bulges, and local buckling of the steel tube was induced after the occurrence of shear cracks. A similar result was reached by Prion and Boehme [10]. For the steel tubular column filled with steel-reinforced high-strength concrete, the strength gradually decreased with an increase in axial deformation after N_u was reached. The steel tubular columns filled with steel-reinforced normal concrete even exhibited strain-hardening characteristics long after the steel yielded. Generally, several bulges occurred on the surface of the steel tube (see Fig. 5(b)), and there was no obviously an existent shear failure plane among the bulges. It can, therefore, be concluded that the steel section can substantially prevent shear cracks in concrete from propagating rapidly.

Curves of axial load N versus mid-height deflection for slender specimens ($L/D > 2.8$) are given in Fig. 6. The deflection of slender specimens was not obvious before the axial load reached 0.6–0.7 N_u. Beyond this load level, the deflection increased gradually. When the load approached N_u, the deflection increased rapidly. The lateral displacement caused secondary moments, and this lead to the column failing by bending rather than by compression.

For specimens HS-A2, HS-B2, HS-C2 and HS-D2, tests were carried out under cyclic loading. Load–axial

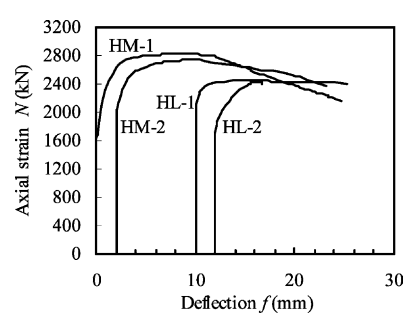

Fig. 6. Curves of load versus deflection at slender column mid-height.

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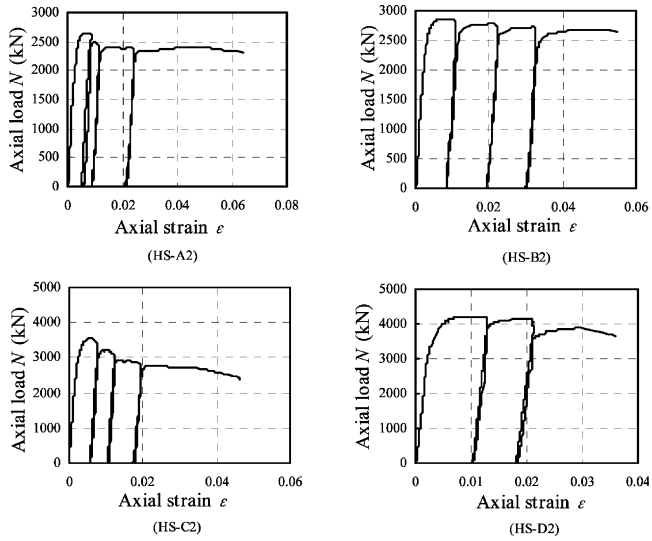

Fig. 7. Load–strain curves under cyclic axial compressive loads.

strain curves of the specimens under cyclic compressive loads are shown in Fig. 7. The results showed that the cyclic behaviour of composite columns was similar to that of an elastoplastic material where the unloading–reloading branches remained linear and parallel, and damage manifested itself only by a progressive plastic strain. The envelope curves of the measured cyclic response practically coincided with the monotonic compressive curves obtained from the analogous specimens as shown in Fig. 4. It was observed that the specimens still possessed adequate strength after several loading–unloading cycles. Thus, the steel tubular columns filled with steel-reinforced concrete have high ductility and capability of absorbing and dissipating energy.

Fig. 8 shows the axial load ratio versus volumetric strain relationships of partial HSC specimens. It can be observed from Fig. 8 that the volumes of specimens decreased with the increase of axial compressive loads when the load ratio was below 0.7. Beyond this load ratio, the volume strain began to increase. The change in volume strain indicated the steel tube began to have confining effect on the concrete and there was a transfer of load from the steel tube to the concrete. The confinement of concrete offered by the steel tube enhanced the strength of concrete and that of the overall column.

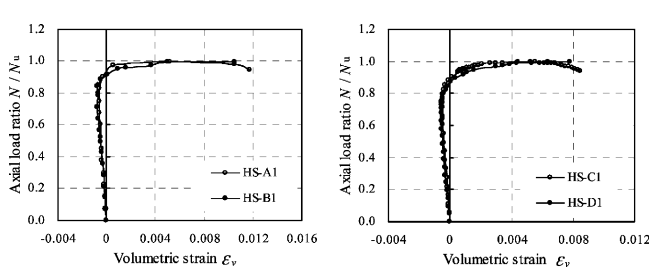

Fig. 8. Load ratio with respect to volumetric strain.

3.2. Effect of four parameters

The steel section and steel tube were the same for specimens NS-A1, NS-A2, HS-A1 and HS-A2 in Fig. 4. For the NC specimens NS-A1 and NS-A2, strain-hardening behaviour occurred after steel yielded resulting in higher strain corresponding to N_u than that of the HSC specimens HS-A1 and HS-A2. This behaviour indicates that the confinement of NC offered by the steel tube is more effective than that of HSC. So, with the enhancement of concrete strength, the strength of composite column increases but its ductility decreases.

The structural steel index was the approximately same for specimens HS-A1, HS-A2, HS-B1, HS-B2, HS-D1 and HS-D2 in Fig. 4. With the increase of θ from 0.376 to 0.733, the ductility of the specimens increased. Fig. 9 shows their average strength of cross-section enhanced as θ increased. It indicates, with the increase of θ, the confining capacity of the steel tube on the concrete increase, and the ductility and strength of composite column increase effectively as well.

Comparing specimens HS-C1 and HS-C2 with specimens HS-D1 and HS-D2 in Fig. 4, it is clear that the ductility and strength of the specimens increased as ρ increased.

The ultimate strengths of specimens HS-B1, HS-B2, HM-1, HM-2, HL-1 and HL-2, which were made by the same material, are plotted against respective L/D ratios in Fig. 10. The trend shows that L/D ratio has a marked detrimental effect upon column strength. N_u decreased as L/D ratios increased from 2.8 to 10. It indicates that the effect of initial imperfections on the strength of the composite columns becomes more and more obvious with the increase of L/D ratios.

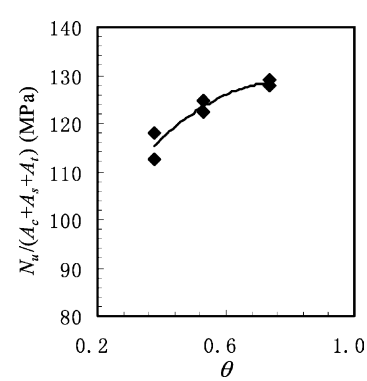

Fig. 9. Average strength–θ relationship.

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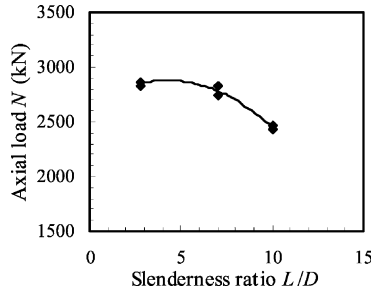

Fig. 10. Effect of slenderness ratio on specimen strength.

4. Calculation of ultimate strength

The experimental study shows that many of the mechanical properties of this type of composite column are consistent with those of conventional CFST columns. Here, calculating methods for CFST columns will be used for reference to deduce the formulas for the evaluation of the ultimate strength of steel tubular columns filled with steel-reinforced concrete under axial compression.

Some design methods for CFST columns have been developed in different countries, such as Manual of Steel Construction, Load and Resistance Factor Design (LRFD) by the American Institute of Steel Construction [11], Building Code Requirements for Reinforced Concrete (ACI 318) by the American Concrete Institute [12], Recommendations for Design and Construction of Concrete-Filled Steel Tubular Structures (AIJ97) by the Architectural Institute of Japan [13], Design of Composite Steel and Concrete Structures (Eurocode 4) by the European Committee for Standardization [14], Steel, Concrete and Composite Bridges (BS5400) by the British Standards Institution [15], Specification for the Design and Construction of Concrete-Filled Steel Tubular Structures (CECS 28:90) by the China Association for Engineering Construction Standardization [16]. In LRFD, ACI 318 and AIJ97, the capacities of the steel and the concrete part of one section are determined separately and then added to each other, therefore the confinement effect of the circular steel tube on the concrete core is not considered and the load-carrying capacity of CFST columns is underestimated [17]. The confinement effect is considered in Eurocode 4 and BS5400. Saw and Liew [18] assessed the design methods for composite columns in LRFD, Eurocode 4 and BS5400, and found that all three methods were mostly conservative when compared with the test results. Huang et al. [19] compared their experimental data with predicted strength obtained by Eurocode 4, and results indicated that the Eurocode 4 might considerably underestimate the ultimate strength of circular CFST columns, particularly for columns with a small diameter-to-thickness ratio (D/t).

In China, experimental and analytical work on CFST columns has been carried out from the 1960s. In analyses, the benefits of confinement for increasing the ultimate strength of CFST columns have been considered. Based on the limit equilibrium method, Cai [20] deduced the following equation for the prediction of the ultimate strength of stub circular CFST columns under axial compression:

$$N_0 = A_c f_c \left(1 + \sqrt{\theta} + \theta\right) \tag{1}$$

Eq. (1) incorporated the influences of f_c and θ on the ultimate strength of stub CFST column, and could predict the ultimate strength of stub circular CFST columns very well [20]. Eq. (1) has been used by CECS 28:90 ($20 \le D/t \le 85\sqrt{235/f_y}$) [16,21].

According to the analysis above, when the steel section is inserted into the CFST column, the ultimate strength of the stub composite column under axial compression can be calculated by the following equation:

$$N_0 = A_c f_c \left(1 + \sqrt{\theta} + \theta + \rho\right) \tag{2}$$

For stub composite columns under axial compressive loads, the column fails due to yielding of the steel and crushing of the concrete under direct compression. But for slender composite columns, the column may fail by elastoplastic or elastic buckling. When the influence of L/D ratio is considered, the ultimate strength of a composite column under axial compression can be calculated by the equation:

$$N_u = \varphi N_0 \tag{3}$$

where φ is the reduction factor due to the slenderness of the column.

Based on the study of Virdi and Dowling [22], the formula for calculating φ of a steel column can be used for a composite column by defining the relative slenderness $\bar{\lambda}$. According to their definition for $\bar{\lambda}$, the following equation can be obtained from Eq. (1):

$$\bar{\lambda} = \frac{L}{\pi}\sqrt{\frac{A_c f_c\left(1 + \theta^{1/2} + \theta + \rho\right)}{E_t I_t + E_s I_s + 0.6 E_c I_c}} \tag{4}$$

in which L is the effective length of the column; E_t, E_s and E_c are the elastic modulus of the steel tube, steel section and concrete, respectively; I_t, I_s and I_c are the second moment of area of the steel tube, steel section and concrete, respectively.

So, according to the shape of the steel cross-section, the coefficient φ can be calculated by the following

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Table 3
Comparison of test and calculated results

Specimen no.	f_{cu} (MPa)	$\bar{\lambda}$	φ	N_u^{Test} (kN)	N_u^{Cal} (kN)	N_u^{EC4} (kN)	N_u^{Test}/N_u^{Cal}	N_u^{Test}/N_u^{EC4}
NS-A1	44.2	0.155	1.000	2350	2124	1916	1.106	1.227
NS-A2	44.2	0.155	1.000	2100	2124	1916	0.989	1.096
NS-B1	44.2	0.153	1.000	2640	2375	2140	1.111	1.234
HS-A1	68.9	0.163	1.000	2700	2542	2269	1.062	1.190
HS-A2	68.9	0.163	1.000	2650	2542	2269	1.042	1.168
HS-B1	68.9	0.161	1.000	2835	2814	2491	1.007	1.138
HS-B2	68.9	0.161	1.000	2862	2814	2491	1.017	1.149
HS-C1	68.9	0.164	1.000	3640	3600	3228	1.011	1.128
HS-C2	68.9	0.164	1.000	3550	3600	3228	0.986	1.100
HS-D1	68.9	0.164	1.000	4130	3977	3597	1.039	1.148
HS-D2	68.9	0.164	1.000	4330	3977	3597	1.089	1.204
HS-E1	68.9	0.158	1.000	2358	2297	1998	1.027	1.180
HM-1	68.9	0.402	0.897	2830	2525	2152	1.121	1.315
HM-2	68.9	0.402	0.897	2735	2525	2152	1.083	1.271
HL-1	68.9	0.574	0.828	2468	2330	1992	1.059	1.239
HL-2	68.9	0.574	0.828	2430	2330	1992	1.043	1.220

equation:

$$\varphi = \begin{cases} 1 & \bar{\lambda} \le 0.2 \\ \dfrac{1}{2\bar{\lambda}^2}\left(1+0.281\sqrt{\bar{\lambda}^2-0.04}+\bar{\lambda}^2 \right. \\ \left. -\sqrt{\left(1+0.281\sqrt{\bar{\lambda}^2-0.04}+\bar{\lambda}^2\right)^2-4\bar{\lambda}^2}\right) & \bar{\lambda} > 0.2 \end{cases} \tag{5}$$

The ultimate strengths of the 16 specimens are calculated using Eqs. (1)–(5). Table 3 lists the values of N_u^{Test}/N_u^{Cal}, where N_u^{Cal} and N_u^{Test} stand for the calculated and test strengths, respectively. The calculated results show good agreement with the test data. The mean value of N_u^{Test}/N_u^{Cal} for 16 specimens is 1.05, with a coefficient of variation of 0.04. For comparison, Table 3 also lists the values of N_u^{Test}/N_u^{EC4}, where N_u^{EC4} stands for the strength predicted using Eurocode 4. The results show that the predicted strengths using Eurocode 4 are on the conservative side and are in reasonable agreement with the test results. The mean value of N_u^{Test}/N_u^{EC4} for 16 specimens is 1.19, with a coefficient of variation of 0.06.

5. Conclusions

This paper has described the test results of steel tubular columns filled with steel-reinforced concrete under axial compression. The influence of f_c, θ, ρ and L/D ratio on the strength and ductility of composite columns has been analysed. The following conclusions are drawn based on these studies.

1. This type of composite column displays the advantages of the concrete-encased steel columns and CFST columns. The strength, ductility and energy absorption of columns is significantly enhanced because of the interaction between the steel tube, steel section and concrete.
2. The steel section can effectively prevent shear cracks in concrete from propagating rapidly. With the increase of ρ, the strength and ductility of the composite column is enhanced.
3. A higher θ leads to a higher confinement effectiveness of the concrete for the composite column. The strength and ductility of the composite column increases with increasing values of θ.
4. The larger L/D ratio, the lower is the strength of the composite column.
5. Expressions for the prediction of the ultimate strength of a composite column under axial compression were proposed, and calculated results are in good agreement with the test results reported in this paper.

Acknowledgements

The work described in this paper was part of a project entitled "Studies on a new design model for heavily loaded steel-concrete composite columns", and funded by the National Natural Science Foundation of China under Grant No. 50078008.

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