JOURNAL OF CIVIL ENGINEERING AND MANAGEMENT
ISSN 1392-3730/eISSN 1822-3605
2015 Volume 21(5): 666–676
10.3846/13923730.2014.893913

THEORETICAL MODEL FOR DOUBLE-SKINNED CONCRETE-FILLED-STEEL-TUBULAR COLUMNS WITH EXTERNAL CONFINEMENT

Chun Xiao DONG[a], Johnny Ching Ming HO[b]

[a]*Department of Civil Engineering, The University of Hong Kong, Pok Fu Lam Road, Hong Kong*
[b]*School of Civil Engineering, The University of Queensland, Brisbane, Australia*

Received 25 Oct 2012; accepted 02 Jan 2013

Abstract. Recent advances in the production of super-fine cement and filler technology has made the production of high-strength concrete (HSC) of 120 MPa practicable in the industry. Nonetheless, the application of such HSC in real construction is still limited. One of the reasons that inhibits the use of HSC is the brittleness, which causes HSC structures to fail explosively if the concrete confinement is not adequate. The traditional method of installing transverse steel as confinement is not feasible in HSC structures, as the steel will be too congested to ensure proper concrete placing. To overcome the problem, double-skinned high-strength concrete-filled-steel-tubular (HSCFST) columns has been advocated, which could provide large, continuous and uniform confinement to HSC. However, a major shortcoming of the double-skinned HSCFST columns is the imperfect interface bonding that occurs at the elastic stage that reduces the elastic strength and stiffness of columns. To improve the situation, the authors have verified previously that using external steel rings on the outer steel tube can successfully restrict the dilation of HSCFST columns and thus restore an intact interface bonding condition. As a continued study, the authors will in this paper develop a theoretical model for predicting the uni-axial load-carrying capacity of doubled-skinned HSCFST columns.

Keywords: columns, concrete-filled, double-skin tubular, external confinement, rings.

Introduction

Because of the recent rapid development of superfine materials such as micro-silica and superfine cement, as well as the matured filler technology (Goldman, Bentur 1993; Haque, Kayali 1998), it is now fairly easy to produce ordinary high-strength concrete (HSC) of compressive strength up to 120 MPa. The advantage of using HSC over normal-strength concrete (NSC) is that it increases considerably the strength-to-weight ratio, which can decrease the required construction materials and demolition wastes. It also saves extra floor space for maximizing usable areas. Furthermore, due to its larger elastic stiffness, it effectively limits the overall and inter-storey drift ratios of tall buildings to within the tolerance of serviceability limit state. However, despite these appealing merits, the application of HSC in column construction is still not very common. The maximum concrete strength that is commonly adopted in practical construction is limited to 60–80 MPa, which is under-utilising the construction materials and wasting available floor space that can possibly be saved.

One of the major reasons that inhibits the use of HSC is its brittleness (Gettu *et al.* 1990; Cusson, Paultre 1994; Marzouk, Chen 1995; Zhou *et al.* 1995; Ho, Zhou 2011), which needs to have substantial confinement to protect the concrete core from explosive failure. The traditional method of utilizing transverse steel in the form of closed hoops or ties for NSC reinforced concrete (RC) columns is not applicable in HSC columns, because the transverse reinforcement required for providing sufficient confinement against explosive failure is too congested to enable good placing quality of concrete. In order to have a breakthrough in the maximum limit of concrete strength that can be practically used in column construction, special type of confinement that can provide larger, more continuous and uniform confining pressure to concrete should be devised. One of the most ineffective reasons for adopting transverse reinforcement in RC columns is the concrete arching action, which decreases the effectively confined concrete area. In order to eliminate this action, a continuous concrete confinement in the form of steel tube has been advocated. Two composite structural forms of high-strength concrete-filled-steel-tubular (HSCFST) column that contains one (i.e. single-skinned) or two hollow steel tubes (i.e. double-skinned) for concrete confinement were put forward.

From structural point of view, these forms of composite column construction can provide larger axial

Corresponding author: Johnny Ching Ming Ho
E-mail: *johnny.ho@uq.edu.au*

Copyright © 2015 Vilnius Gediminas Technical University (VGTU) Press
www.tandfonline.com/tcem
Taylor & Francis
Taylor & Francis Group

strength (Wright *et al.* 1991; Wei *et al.* 1995; Zhao, Grzebieta 2002; Zhao *et al.* 2002; Giakoumelis, Lam 2004; Tao *et al.* 2004; Young, Ellobody 2006; Dabaon *et al.* 2009; Kuranovas *et al.* 2009; de Oliveira *et al.* 2010; Szmigiera *et al.* 2010), bending stiffness, moment capacity (Elchalakani *et al.* 2001; Lin, Tsai 2001; Zhao, Grzebieta 2002; Chitawadagi, Narasimhan 2009; Lu *et al.* 2009), better ductility (Kitada 1998; Schneider 1998; Elremaily, Azizinamini 2002; Zhao, Grzebieta 2002; Lin, Tsai 2001) and excellent seismic performance (Varma *et al.* 2002; Sakino *et al.* 2004; Yang, Han 2008; Lu *et al.* 2010; Zhao *et al.* 2010; Montejo *et al.* 2012). From cost effectiveness point of view, the tubes act as both the longitudinal reinforcement that reduces the cost for steel bars fixing and formwork that reduces the cost for formwork fabrication. From environmental point of view, the size of HSCFST column can be up to 50% smaller than that of HSC columns providing the same load-carrying capacity. It successfully decreases the embodied energy level in the building structures. Compared with single-skinned HSCFST columns, double-skinned HSCFST columns further improve the strength-to-weight ratio by replacing the bulky central concrete with an inner steel tube with smaller cross-sectional area. It also provides a dry atmosphere within the inner steel tube, which is particularly useful to house sub-sea oil production facilities for offshore structures (Shakir-Khalil 1991; Yang *et al.* 2008; Zhao *et al.* 2010). For other structures, the dry environment is also useful for accommodation of conduits, drainage and maintenance check purposes.

However, one major problem (Wei *et al.* 1995; Huang *et al.* 2011) for the HSCFST columns is the imperfect steel-concrete interface bonding that occurs in the elastic stage since steel dilates more than concrete (Köster, Franz 1961; Persson 1999; Ferretti 2004; Lu, Hsu 2007). Consequently, the confinement is not activated in the initial elastic stage, which decreases the elastic strength and stiffness. In order to resolve this problem, the authors have previously proposed to use external confinement in the form of steel rings to restrict the dilation of HSCFST columns for restoring an intact interface bonding (Dong, Ho 2012; Ho, Luo 2012; Lai, Ho 2014). A preliminary test programme has been carried out and the results showed that the Poisson's ratios of columns can be successfully reduced to close to 0.2, which is that of plain concrete. It verified that the concrete would be in perfect bond with the steel skin under the extra confinement effect provided by the steel rings. In the past, some efforts had been spent on installing internal stiffeners and binding bars for achieving similar purpose (Huang *et al.* 2002; Tao *et al.* 2005; Cai, He 2006; Tao *et al.* 2007). However, it is worth noting that the welding of these internal stiffeners and binding bars is more difficult than that of the proposed external confinement, in particular for doubled-skinned HSCFST columns where the gap between steel skins is very limited.

In order to investigate the behaviour of unconfined and ring-confined double-skinned HSCFST column, it is necessary to develop a numerical model for evaluating the uni-axial load carrying capacity of double-skinned CFST column. Previously, some research work has been done on the theoretical model for double-skinned HSCFST columns (Han *et al.* 2004; Tao *et al.* 2004; Huang *et al.* 2010; Tan, Zhang 2010; Uenaka *et al.* 2010; Zhao *et al.* 2010; Hu, Su 2011). However, there was no such model proposed for double-skinned HSCFST columns containing external confinement. Furthermore, there is no design model and guidelines provided in Eurocode 4 (EC4 2004) for strength prediction of externally confined double-skinned HSCFST columns. Therefore, the design according to the Eucocode 4 will be very conservative. To fill up the gap, the authors will in this paper derive a theoretical model for evaluating the uni-axial load-carrying capacity of double-skinned HSCFST columns based on the results obtained for twenty double-skinned HSCFST columns in a previous experimental programme. For verification, the proposed model will be used to calculate the theoretical strength of double-skinned HSCFST columns tested by the authors as well as other researchers.

1. Details of specimens

In a previous study conducted by the authors (Dong, Ho 2012), a total of twenty concrete-filled double-skin tubular columns have been fabricated and tested under uni-axial compression load. The specimens were divided into four groups based on the provision of confinement, concrete cylinder strength and the hollow ratio χ (defined as $\chi = \dfrac{D_i}{D_o - 2t_o}$), where D_i and D_o are the diameters of the inner and outer tubes respectively, t_o is the thickness of the outer tube: (1) Four double-skinned CFST columns of hollow ratio 0.56 and external steel rings of various spacing ($5t_o$, $10t_o$, $15t_o$ and $20t_o$); (2) four double-skinned CFST columns of hollow ratio 0.72 and external steel rings of various spacing ($5t_o$, $10t_o$, $15t_o$ and $20t_o$); (3) one double-skinned CFST column of hollow ratio 0.56 but without external steel rings; (4) one double-skinned CFST column of hollow ratio of 0.72 but without external steel rings. In each of the two sets of specimens (i.e. χ = 0.56 and 0.72), concrete cylinder strength of 50 and 85 MPa were adopted. The grade of both inner and outer steel tubes was S355 produced as per BS EN 10210-2: 2006 (2006). For all tested specimens, the thickness of both inner and outer steel tubes was 5 mm and the diameter of outer steel tube was 168.3 mm (measured to the outer face). For columns of hollow ratios 0.56 and 0.72, the diameters of inner tubes were 88.9 and 114.3 mm respectively (measured to the outer face). The total height of the specimens was 330 mm (aspect ratio of 2). The section and material properties of the specimens are summarised in Table 1.

The external steel rings were made of mild steel round bars of 8 mm diameter and the yield strength was $f_{y,R}$ = 300 MPa. The rings were welded to the outer tubes at different spacing and the lap length was ten times the

Table 1. Details of specimens and materials' properties

Specimen label	D_i (mm)	t_i (mm)	$f_{y,i}$ (MPa)	D_o (mm)	t_o (mm)	$f_{y,o}$ (MPa)	f'_c (MPa)	$f_{y,R}$ (MPa)
D-0.56-50-5	88.9	5	450	168.3	5	360	50	300
D-0.56-50-10	88.9	5	450	168.3	5	360	50	300
D-0.56-50-15	88.9	5	450	168.3	5	360	50	300
D-0.56-50-20	88.9	5	450	168.3	5	360	50	300
D-0.56-50-0	88.9	5	450	168.3	5	360	50	N/A
D-0.56-85-5	88.9	5	450	168.3	5	360	85	300
D-0.56-85-10	88.9	5	450	168.3	5	360	85	300
D-0.56-85-15	88.9	5	450	168.3	5	360	85	300
D-0.56-85-20	88.9	5	450	168.3	5	360	85	300
D-0.56-85-0	88.9	5	450	168.3	5	360	85	N/A
D-0.72-50-5	114.3	5	430	168.3	5	360	50	300
D-0.72-50-10	114.3	5	430	168.3	5	360	50	300
D-0.72-50-15	114.3	5	430	168.3	5	360	50	300
D-0.72-50-20	114.3	5	430	168.3	5	360	50	300
D-0.72-50-0	114.3	5	430	168.3	5	360	50	N/A
D-0.72-85-5	114.3	5	430	168.3	5	360	85	300
D-0.72-85-10	114.3	5	430	168.3	5	360	85	300
D-0.72-85-15	114.3	5	430	168.3	5	360	85	300
D-0.72-85-20	114.3	5	430	168.3	5	360	85	300
D-0.72-85-0	114.3	5	430	168.3	5	360	85	N/A

diameter of the steel bar (80 mm). Each ring was welded to the outer tube at eight locations with a central angle of 45° separated from each other. Figures 1 and 2 show the test setup and the details of the specimens.

A naming system consisting of one letter and three numbers has been used to represent the specimens. For instance, 'D-0.72-50-5' represents a double-skinned CFST column (indicated by the first letter "D"), a hollow ratio of 0.72 (indicated by the first number "0.72"), a concrete cylinder strength of about 50 MPa on the testing day (indicated by the second number "50") and lastly five times the thickness of the outer steel tube as the ring spacing (indicated by the last number "5"). Alternatively, 'D-0.56-85-0' represents a double-skinned CFST column (indicated by the first letter "D") with a hollow ratio of

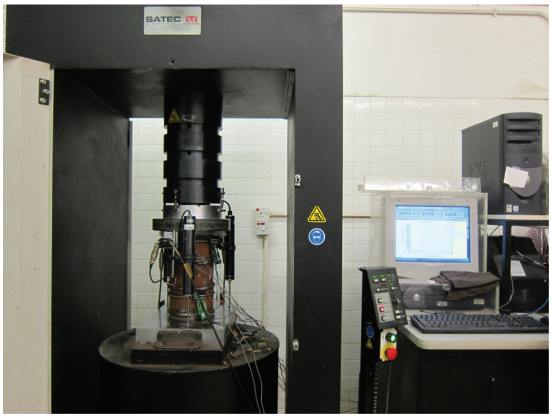
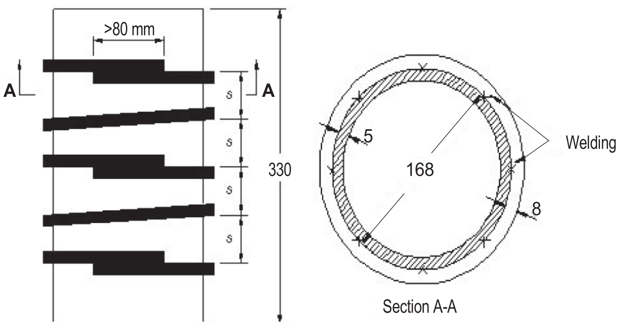


Fig. 1. Test setup



s	mm	No. of rings
5t	25	13
10t	50	7
15t	75	5
20t	100	3

Fig. 2. Details of external steel rings of specimens

0.56 (indicated by the first number "0.56"), a concrete cylinder strength of about 85 MPa on the testing day (indicated by the second number "85"), and lastly no external steel ring (i.e. zero spacing indicated by the last number "0").

2. Theoretical model for confined double-skinned CFST columns

A lot of research has been conducted on the theoretical models of double-skinned HSCFST columns (Tao, Han 2006; Tan, Zhang 2010; Uenaka *et al.* 2010; Hu, Su 2011). Nevertheless, the theoretical models were mainly for columns without external confinement. Therefore, the application of such models will underestimate the strength capacity of double-skinned HSCFST columns installed with external confinement. Furthermore, the existing Eurocode 4 (EC4 2004) is not suitable for designing confined double-skinned HSCFST columns because no guidelines are provided to account for the enhanced strength of concrete and steel tube due to the provision of external confinement. Therefore in this paper, the authors will propose a theoretical model for predicting the uni-axial strength of confined double-skinned HSCFST columns. The validity of the theoretical model will be verified by comparing the theoretical results with the experimental results obtained previously by the authors.

The formulas for evaluating the axial capacity of the double-skinned CFST columns with/without external confinement are shown as follows:

$$N_p = N_i + N_o + N_{cc}; \quad (1)$$

$$N_i = f_i A_i; \quad (2a)$$

$$N_o = f_o A_o; \quad (2b)$$

$$N_{cc} = f_{cc} A_{cc}, \quad (2c)$$

where N_p, N_i, N_o and N_{cc} in Eqn (1) represent the predicted axial load capacity of double-skinned CFST column with/without external confinement, axial load sustained by the inner steel tube, outer steel tube and core concrete respectively. In Eqn (2), f_i and f_o are the axial stresses in the inner and outer tubes respectively under bi-axial stress state; f_{cc} is the axial stress in the core concrete under the confining pressure f_r. A_i, A_o and A_{cc} are the cross-section areas of the inner tube, outer tube and the core concrete respectively. In this model, it is assumed that the confining pressure is uniform along the total height of the specimens. The confining pressure provided by the inner and outer steel tubes is the same based on force equilibrium. The radial stresses in the steel tubes are negligible due to the large contact area. Figure 3(a) shows the free body diagram of the outer steel tube together with the external steel rings, if any, subjected to confining pressure f_r. The hoop tensile stress developed in the outer tube can

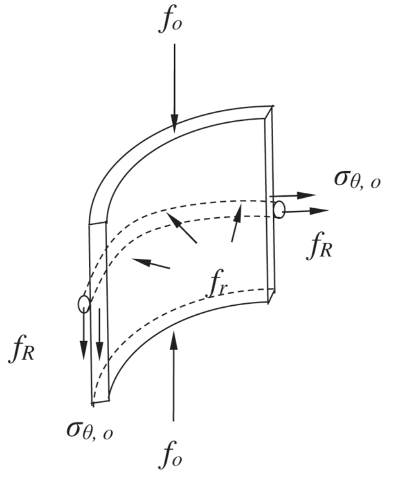


Fig. 3(a). Free body diagram of the outer tube

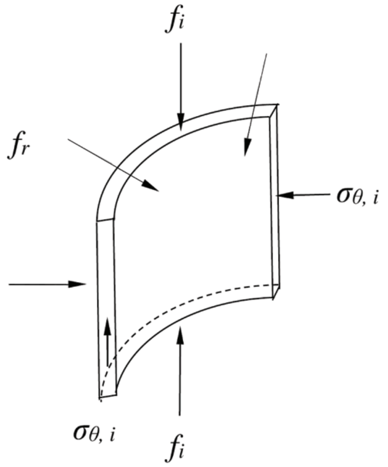


Fig. 3(b). Free body diagram of the inner tube

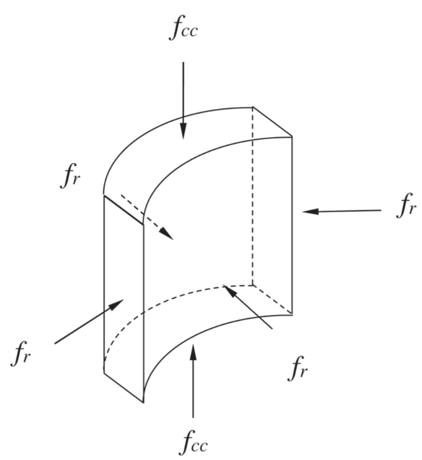


Fig. 3(c). Free body diagram of the core concrete be derived by considering the force equilibrium of the outer tube under the confining pressure provided by the concrete and external steel rings. The hoop tensile stress $\sigma_{\theta,o}$ for ring-confined specimens is shown in Eqn (3a):

$$\sigma_{\theta,o} = \frac{-f_r(D_o - 2t_o)h + 2nA_R\sigma_R}{2t_o h}, \quad (3a)$$

where: $\sigma_{\theta,o}$ is the hoop tensile stress developed in the outer tube; σ_R is the tensile stress acting in the external rings; A_R is the cross-section area of each steel ring; D_o and t_o represent the diameter and thickness of the outer steel tube; h stands for the total height of the column; n is the number of the external steel rings welded on the specimen. For unconfined double-skinned CFST column, take $n = A_R = 0$, and Eqn (3a) is simplified to:

$$\sigma_{\theta,o} = -\frac{f_r(D_o - 2t_o)}{2t_o}. \quad (3b)$$

Figure 3(b) shows the free body diagram of the inner steel tube subjected to confining pressure f_r. The hoop compressive stress developed in the inner tube is denoted by $\sigma_{\theta,i}$. By considering the force equilibrium of the inner tube, Eqn (4) can be established as follows:

$$\sigma_{\theta,i} = \frac{f_r D_i}{2t_i}, \quad (4)$$

where: $\sigma_{\theta,i}$ is the hoop compressive stress developed in the inner tube; D_i and t_i represent the diameter and thickness of the inner steel tube. Under bi-axial stress state, the yield strength of the outer and inner tubes is not be the same as the uni-axial yield strength. Their respective yield strength at bi-axial stress state can be determined by the von Mises yield criterion. Eqns (5a) and (5b) show the von Mises yield criterion for inner and outer steel tube, respectively:

$$f_o^{\,2} - f_o\sigma_{\theta,o} + \sigma_{\theta,o}^{\,2} = f_{y,o}^{\,2}; \quad (5a)$$

$$f_i^2 - f_i\sigma_{\theta,i} + \sigma_{\theta,i}^{\,2} = f_{y,i}^{\,2}, \quad (5b)$$

where $f_{y,o}$, $f_{y,i}$ are the uni-axial yield strength of the steel tubes. To find out the values of the yield stresses of the outer and inner tubes, it is necessary to determine the hoop stress $\sigma_{\theta,o}$ of the outer tube at yielding. Previously, Hatzigeorgiou (2008) reported that a major parameter that affects the hoop stress is the diameter-to-thickness ratio. The hoop stress is correlated to the uni-axial yield strength by a new parameter α_θ, which depends on the diameter-to-thickness ratio and yield strength of steel tube. The formulas proposed by Hatzigeorgiou (2008) relating the hoop stress $\sigma_{\theta,o}$ of the outer tube and the diameter-to-thickness ratio (D_o/t_o) and yield strength ($f_{y,o}$) are expressed in Eqn (6):

$$\sigma_{\theta,o} = \alpha_\theta f_{y,o}; \quad (6a)$$

$$\alpha_\theta = \exp(\ln(\frac{D_0}{t_0}) + \ln(f_{y,o}) - 11) \leq 1. \quad (6b)$$

By calculating the hoop stress at yielding, the yield strength of steel tubes under bi-axial state, as well as the confining pressure f_r, can be determined. The strength calculated can then be substituted back into Eqns (2a) and (2b) to obtain the compressive stresses in the outer and inner tubes. Normally, in order to fully utilise the yield strength of the steel tubes to confine the in-filled concrete, the steel tube will be designed to be sufficiently strong that no stability failure would occur before the steel tubes yield and concrete crushes. This can be verified by checking the buckling strain ε_{lb} of the steel tube as shown in Eqn (7) (O'Shea, Bridge 1997):

$$\varepsilon_{lb} = \varepsilon_y(0.2139R^{-1.413}); \quad (7a)$$

$$R = \frac{D_o}{t_o}\frac{f_{y,o}}{E_{so}}, \quad (7b)$$

where: ε_y is the yield strain of the steel tube under uni-axial compression; and E_{so} is the elastic modulus of steel tube. In order to verify that the steel tube can reach the yield strength before any instability occurs, it needs to check that $\varepsilon_{lb} \geq \varepsilon_y$ (or $R \leq 0.336$). Otherwise, the stress in the steel tube should be taken as $\varepsilon_{lb}E_{so}$.

For the compression force in the confined concrete, the confined concrete strength can be assessed by the Eqn (8) proposed by Mander and Priestley (1988). Figure 3(c) shows the free body diagram of core concrete.

$$f_{cc} = f_c' + 4.1f_r, \quad (8)$$

where f_c' is unconfined concrete cylinder strength.

The following summarises the procedure to determine the uni-axial strength of a double-skinned CFST column with/without confinement:

1. Determine the hoop stress of the outer steel tube $\sigma_{\theta,o}$ by Eqn (6);
2. Determine the confining pressure f_r by substituting $\sigma_{\theta,o}$ into Eqn (3a) and take s$_R$ equal to the yield strength of ring $f_{y,R}$ assuming that steel rings yield when the column attains uni-axial strength;
3. Calculate the yield strength of the outer tube f_o and inner tube f_i under bi-axial state by Eqns (5a) and (5b) respectively;
4. Calculate the confined concrete strength f_{cc} by substituting f_r into Eqn (8);
5. Calculate the uni-axial load-carrying capacity of double-skinned CFST columns by substituting f_i, f_o and f_{cc} into Eqns (2a), (2b) and (2c) respectively;
6. Check the stability of the column by verifying that the buckling strain ε_{lb} is larger than the yield strength of steel tube ε_y. Otherwise, the stresses in

the outer and inner tube should be taken as the buckling strain multiplied by the elastic modulus.

3. Verification with test results

3.1. Verification of ring-confined double-skinned CFST columns

The theoretical axial load-carrying capacities of the tested ring-confined double-skinned CFST columns specimens evaluated by the proposed analytical model are listed in Table 2(a). The theoretical results were also compared with the predicted uni-axial strength as per Eurocode 4 Part 1-1 (EC4 2004) in the same table. It is worth noting that the formula in Eurocode 4 does not take into account the effect of external confinement.

In the table, N_p represents the predicted load-carrying capacity evaluated by authors' model, N_t represents the experimentally measured load-carrying capacity and N_{EC} represents the load-carrying capacity evaluated using Eurocode 4. From the table, it is observed that:

1. The average N_p/N_t ratios for ring-confined double-skinned CFST columns with hollow ratio of 0.56 and concrete strength of 50 and 85 MPa are 0.923 and 0.964 respectively. The maximum difference between the theoretical and test results is 8.3% underestimation.
2. The average N_p/N_t ratios for ring-confined double-skinned CFST columns with hollow ratio of 0.72 and concrete strength of 50 and 85 MPa are 0.903 and 0.965 respectively. The maximum difference between the theoretical and test results is 2.3% overestimation and 15.9% underestimation.
3. The average N_p/N_t ratio for all ring-confined double-skinned CFST columns is 0.940. Comparing with the average N_{EC}/N_t ratio for all ring-confined double-skinned CFST columns, which is 0.815, it is evident that the proposed model can predict much more accurately the load-carrying capacity of ring-confined double-skinned CFST columns by taking into account the confinement effect provided by the external rings.

3.2. Verification of unconfined double-skinned CFST columns

The analytical model has also been applied to evaluate the theoretical axial load-carrying capacity of unconfined double-skinned CFST columns tested by the authors and by other previous researchers (Wei *et al.* 1995; Tao *et al.* 2004; Lin, Tsai 2001; Uenaka *et al.* 2010; Han *et al.* 2011). Apart from comparing with the experimental results, the theoretical results were also compared with the

Table 2(a). Comparison between theoretical and experimental results of ring-confined double-skinned CFST columns

Specimen Label	N_p (kN)	N_{EC} (kN)	N_t (kN)	$\frac{N_{EC}}{N_t}$	$\frac{N_p}{N_t}$
D-0.56-50-5	3212	2474	3464	0.714	0.927
D-0.56-50-10	2878	2474	3107	0.796	0.926
D-0.56-50-15	2723	2474	2971	0.833	0.917
D-0.56-50-20	2639	2474	– #	– #	– #
Average				0.781	0.923
D-0.56-85-5	3684	2934	3788	0.775	0.973
D-0.56-85-10	3350	2934	3394	0.864	0.987
D-0.56-85-15	3194	2934	3434	0.854	0.930
D-0.56-85-20	3109	2934	3222	0.911	0.965
Average				0.851	0.964
D-0.72-50-5	3005	2280	3209	0.711	0.936
D-0.72-50-10	2620	2280	2873	0.794	0.912
D-0.72-50-15	2493	2280	2964	0.769	0.841
D-0.72-50-20	2480	2280	2688	0.848	0.923
Average				0.781	0.903
D-0.72-85-5	3339	2605	3265	0.798	1.023
D-0.72-85-10	2950	2605	3049	0.854	0.968
D-0.72-85-15	2823	2605	3037	0.858	0.930
D-0.72-85-20	2809	2605	2995	0.870	0.938
Average				0.845	0.965
(Overall) Average				0.815	0.940
(Overall) Standard deviation				0.058	0.041
(Overall) Maximum (Overestimation)				N/A	1.023
(Overall) Minimum (Underestimation)				0.711	0.841

Result is NOT included because of poor concrete compaction.

Table 2(b). Comparison between theoretical and experimental results of unconfined double-skinned CFST columns

Specimen label	N_{p1} (kN)	N_{p2} (kN)	N_{p3} (kN)	N_{p4} (kN)	N_{EC} (kN)	N_p (kN)	N_t (kN)	$\frac{N_{p1}}{N_t}$	$\frac{N_{p2}}{N_t}$	$\frac{N_{p3}}{N_t}$	$\frac{N_{p4}}{N_t}$	$\frac{N_{EC}}{N_t}$	$\frac{N_p}{N_t}$	Refe-rences
D-0.56-50-0	2099	2486	2848	2417	2474	2561	2852	0.736	0.872	0.999	0.847	0.867	0.898	Authors' test results
D-0.56-85-0	2517	2965	3320	2856	2934	3032	3218	0.782	0.873	1.032	0.888	0.912	0.942	
D-0.72-50-0	1484	2335	2354	2205	2280	2379	2674	0.555	0.921	0.88	0.825	0.853	0.890	
D-0.72-85-0	1727	2669	2684	2513	2605	2709	2994	0.577	0.892	0.896	0.839	0.870	0.905	
A1-1	198	272	239	261	269	277	283	0.700	–**	–**	0.922	0.951	0.979	(Wei *et al.* 1995)
A1-2	200	259	229	250	258	265	285	0.702	–**	–**	0.877	0.905	0.930	
A2-1	153	322	278	308	316	330	348	0.440	–**	–**	0.885	0.908	0.948	
A2-2	175	308	269	295	303	312	348	0.503	–**	–**	0.848	0.871	0.897	
A3-1	133	365	310	346	353	365	395	0.337	–**	–**	0.876	0.894	0.924	
A3-2	139	367	310	347	355	367	395	0.352	–**	–**	0.878	0.899	0.929	
B1-1	313	317	306	306	323	332	330	0.948	0.960	–**	0.927	0.979	1.006	
B1-2	309	308	298	298	314	323	335	0.922	0.918	–**	0.890	0.937	0.964	
B2-1	322	358	339	344	363	374	386	0.834	0.927	–**	0.891	0.940	0.969	
B2-2	328	363	346	348	366	377	395	0.830	0.918	–**	0.881	0.927	0.954	
C1-1	361	345	354	334	359	368	378	0.955	0.911	–**	0.884	0.950	0.974	
C1-2	359	340	350	330	354	363	385	0.932	0.882	–**	0.857	0.919	0.943	
C2-1	386	383	393	369	395	406	432	0.894	0.887	0.910	0.854	0.914	0.940	
C2-2	385	376	387	362	388	398	408	0.944	0.921	0.949	0.887	0.951	0.975	
D1-1	307	280	273	278	292	275	283	1.085	–**	–**	0.982	1.032	0.972	
D2-1	258	255	227	252	256	261	299	0.863	–**	–**	0.843	0.856	0.873	
D3-1	332	309	298	306	321	325	357	0.930	–**	–**	0.857	0.899	0.910	
D4-1	378	344	352	341	364	369	380	0.995	0.905	–**	0.897	0.958	0.971	
D5-1	420	384	425	381	416	426	443	0.948	0.866	0.959	0.860	0.939	0.962	
D6-1	535	525	586	503	631	648	644	0.831	0.816	0.910	0.781	0.980	1.006	
E1-1	320	323	324	310	350	352	357	0.896	0.904	–**	0.868	0.980	0.986	
E2-1	439	428	463	414	455	451	477	0.920	0.897	0.971	0.868	0.954	0.945	
E3-1	372	373	364	362	385	386	417	0.892	0.894	–**	0.868	0.923	0.926	
E4-1	588	571	645	555	619	613	598	0.983	0.955	1.079	0.928	1.035	1.025	
E5-1	523	508	537	496	541	538	551	0.949	0.922	0.975	0.900	0.982	0.976	
E6-1	471	473	455	463	487	490	524	0.899	–**	–**	0.884	0.929	0.935	
cc2a	1552	1518	2208	1881	1631	1641	1790	0.867	0.848	1.234	1.051	0.911	0.917	(Tao *et al.* 2004)
cc2b	1552	1518	2208	1881	1631	1641	1791	0.867	0.848	1.233	1.050	0.911	0.916	
cc3a	1521	1474	1869	1668	1548	1568	1648	0.923	0.894	1.134	1.012	0.939	0.951	
cc3a	1521	1474	1869	1668	1548	1568	1650	0.922	0.893	1.133	1.011	0.938	0.950	
cc4a	1151	1242	1228	1252	1229	1220	1435	0.802	–**	–**	0.872	0.856	0.850	
cc4b	1151	1242	1228	1252	1229	1220	1358	0.848	–**	–**	0.922	0.905	0.898	
cc5a	753	795	978	812	808	809	904	0.833	0.880	1.082	0.898	0.894	0.895	
cc5b	753	795	978	812	808	809	898	0.839	0.885	1.089	0.904	0.900	0.901	
cc6a	2320	2208	2867	2629	2339	2419	2421	0.958	0.912	1.184	1.086	0.966	0.999	
cc6b	2320	2208	2867	2629	2339	2419	2460	0.943	0.897	1.165	1.069	0.951	0.983	
cc7a	3253	3048	3802	3403	3189	3377	3331	0.977	0.915	1.141	1.022	0.957	1.014	
cc7b	3253	3048	3802	3403	3189	3377	3266	0.996	0.933	1.164	1.042	0.976	1.034	
DS-2	2417	2331	2627	2222	2981	2530	2750	0.879	0.848	0.955	0.808	1.084	0.920	
DS-6	2011	1867	2087	1823	2302	2098	2311	0.870	0.808	0.903	0.789	0.996	0.908	
c10-375	460	435	580	2139	518	578	635	0.724	0.686	0.913	3.369	0.816	0.910	(Uenaka *et al.* 2010)
c10-750	433	405	485	1229	470	518	543	0.797	0.745	0.893	2.263	0.866	0.954	
c10-1125	360	330	342	328	363	387	378	0.952	0.872	–**	0.868	0.960	1.024	
c16-375	647	603	889	1088	744	847	852	0.759	0.708	1.043	1.277	0.873	0.994	
c16-750	651	595	738	751	700	785	728	0.894	0.817	1.014	1.032	0.962	1.078	
c16-1125	606	548	550	534	594	646	589	1.029	0.930	–**	0.907	1.008	1.097	
c23-375	738	701	1057	950	867	905	968	0.762	0.724	1.092	0.981	0.896	0.935	
c23-750	758	708	883	791	829	869	879	0.862	0.805	1.005	0.900	0.943	0.989	
c23-1125	712	666	652	644	710	758	704	1.011	0.946	–**	0.915	1.009	1.077	
C1-1	2268	2350	2498	2391	2489	2622	2537	0.894	0.926	–**	0.942	0.981	1.034	(Han *et al.* 2011)
C1-2	2268	2350	2498	2391	2489	2622	2566	0.884	0.916	–**	0.932	0.970	1.022	
C2-1	2810	2727	3476	3067	3071	3200	3436	0.818	0.794	1.012	0.893	0.894	0.931	
C2-2	2810	2727	3476	3067	3071	3200	3506	0.801	0.778	0.991	0.875	0.876	0.913	
Average								0.839	0.872	1.030	0.982	0.932	0.957	
Standard deviation								0.157	0.065	0.106	0.378	0.051	0.052	
Maximum (Overestimation)								1.085	0.960	1.234	3.369	1.084	1.097	
Minimum (Underestimation)								0.337	0.686	0.880	0.781	0.816	0.850	

** The respective equation is not applicable.

Table 3. Existing models for unconfined double-skinned CFST columns

Proposer(s)	Formulas	Limitation(s)
Tan and Zhang (2010)	$N = Af_{ssc}$ $f_{ssc} = (1.212 + a\xi_{ssc} + b\xi_{ssc}^2)f_{ck}$ $a = 0.1759 f_{ss}/235 + 0.974$ $b = -0.1038 f_{ck}/20 + 0.0309$ $f_{ss} = \frac{A_{so}f_{so} + A_{si}f_{si}}{A_{so} + A_{si}}$ $\quad \xi_{ssc} = \frac{\sum A_s f_s}{A_c f_{ck}}$	
Tao *et al.* (2004)	$N_u = N_{osc,u} + N_{i,u}$ $N_{osc,u} = f_{scy} A_{sco}$ $\quad A_{sco} = A_{so} + A_c$ $f_{scy} = C_1\chi^2 f_{syo} + C_2(1.14 + 1.02\xi)f_{ck}$ $C_1 = \alpha/(1+\alpha)$ $\quad C_2 = (1 + \alpha_{no\min al})/(1+\alpha)$ $\alpha = A_{so}/A_c$ $\quad \alpha_{no\min al} = A_{so}/A_{c,no\min al}$ $\xi = \frac{A_{so}f_{syo}}{A_{c,no\min al} f_{ck}}$	Only applicable to unconfined double-skinned CFST columns with hollow section ratio less than 0.8
Uenaka *et al.* (2010)	$N_{u,CFDST} = \left[2.86 - 2.59(\frac{D_i}{D_o})\right]A_{so}f_{so} + A_{si}f_{si} + A_c f_c'$	Only applicable to unconfined double-skinned CFST columns with D_i/D_o ratio between 0.2 and 0.7
Hu and Su (2011)	$P_n = P_o\left[0.658^{(P_o/P_e)}\right]$ if $P_e \geq 0.44P_o$ $P_n = 0.877P_e$ if $P_e < 0.44P_o$	

analytical model proposed by other researchers (Tao, Han 2006; Tan, Zhang 2010; Uenaka *et al.* 2010; Hu, Su 2011) and the strength predicted by Eurocode 4. The comparison is summarised in Table 2(b). The existing models and their respective limitations, if any, are listed in Table 3.

In the table, N_{p1}, N_{p2}, N_{p3} *and* N_{p4} represent the predicted load-carrying capacities evaluated by Tan and Zhang (2010), Tao *et al.* (2004), Uenaka *et al.* (2010) and Hu and Su (2011), respectively. N_p is the predicted load-carrying capacity evaluated by authors' proposed model, N_t is the measured load-carrying capacity and N_{EC} is the load-carrying capacity evaluated using Eurocode 4. From the table, it is observed that:

1. The average N_{p1}/N_t, N_{p2}/N_t, N_{p3}/N_t *and* N_{p4}/N_t ratios for all unconfined double-skinned CFST columns are 0.839, 0.872, 1.030 and 0.982 respectively. The maximum differences between the predicted load-carrying capacities by various researchers and the measured values are 23.4% overestimation (Uenaka *et al.* 2010) and 66.3% underestimation (Tan, Zhang 2010).

2. The average N_p/N_t ratio for all unconfined double-skinned CFST columns are 0.957 and the standard deviation is 0.052. The maximum differences between the theoretical and test results are 9.7% overestimation and 15% underestimation.

3. Comparing with the average N_{EC}/N_t ratio for all unconfined double-skinned CFST columns, which is 0.932, it is evident that the proposed model can predict more accurately the load-carrying capacity of unconfined double-skinned CFST columns.

From the above, it is seen that the proposed model predicts fairly accurately the uni-axial load-carrying capacity of both ring- and un-confined double-skinned CFST columns. In particular, the proposed model can predict more accurately the axial load-carrying capacity of ring-confined CFST columns than the Eurocode does.

Conclusions

A new method for providing external confinement to restrict the lateral dilation of double-skinned CFST columns has been proposed and verified previously by uni-axial compression test to have successfully restored the intact interface steel-concrete interface bonding during the elastic stage. Based on the test results, a theoretical model has been built up that takes into account the confining effects provided by both steel rings and steel tube for predicting the uni-axial strength of double-skinned CFST columns. The uni-axial load-carrying capacity evaluated by the proposed theoretical model was compared with the experimental results obtained by the authors and other researchers' and it was found that:

1. For ring-confined double-skinned CFST columns with hollow ratio of 0.56 and 0.72, the theoretical results are in good agreement with the experimental results.

2. For unconfined double-skinned CFST columns with hollow ratios ranging from 0.24 to 0.88, the theoretical model predicts more accurately the load carrying capacity than Eurocode does.

From the comparison, it is evident that the proposed model can predict the uni-axial load carrying capacity of the unconfined and ring-confined double-skinned CFST columns fairly accurately. The validity of the proposed model needs to be further verified by extensive experimental data of confined double-skinned CFST columns.

Acknowledgements

The work described in this paper has been substantially supported by a grant from the Research Grants Council of the Hong Kong Special Administrative Region, China (Project No. HKU 712310E). Technical supports for the experimental tests provided by the laboratory staff of the Department of Civil Engineering, The University of Hong Kong, are gratefully acknowledged.

References

BS EN. 10210-2: 2006. Hot finished structural hollow sections of non-alloy and fine grain steels-Part 2: Tolerances, dimensions and sectional properties. London: British Standards Institute, 2006.

Cai, J.; He, Z. Q. 2006. Axial load behavior of square CFT stub column with binding bars, *Journal of Constructional Steel Research* 62(5): 472–483. http://dx.doi.org/10.1016/j.jcsr.2005.09.010

Chitawadagi, M. V.; Narasimhan, M. C. 2009. Strength deformation behaviour of circular concrete filled steel tubes subjected to pure bending, *Journal of Constructional Steel Research* 65(8): 1836–1845. http://dx.doi.org/10.1016/j.jcsr.2009.04.006

Cusson, D.; Paultre P. 1994. High-strength concrete columns confined by rectangular ties, *Journal of Structural Engineering* 120(3): 783–804. http://dx.doi.org/10.1061/(ASCE)0733-9445(1994)120:3(783)

Dabaon, M.; El-Khoriby, S.; El-Boghdadi, M.; Hassanein, M. F. 2009. Confinement effect of stiffened and unstiffened concrete-filled stainless steel tubular stub columns, *Journal of Constructional Steel Research* 65(8): 1846–1854. http://dx.doi.org/10.1016/j.jcsr.2009.04.012

de Oliveira, W. L. A.; De Nardin, S.; El Debs, A. L. H.; El Debs, M. K. 2010. Evaluation of passive confinement in CFT columns, *Journal of Constructional Steel Research* 66(4): 487–495 http://dx.doi.org/10.1016/j.jcsr.2009.11.004.

Dong, C. X.; Ho, J. C. M. 2012. Uni-axial behaviour of normal-strength CFDST columns with external steel rings, *Steel and Composite Structures* 13(6): 587–606. http://dx.doi.org/10.12989/scs.2012.13.6.587

Elchalakani, M.; Zhao, X. L.; Grzebieta, R. 2001. Concrete-filled circular steel tubes subjected to pure bending, *Journal of Constructional Steel Research* 57(11): 1141–1168. http://dx.doi.org/10.1016/S0143-974X(01)00035-9

Elremaily, A.; Azizinamini, A. 2002. Behavior and strength of circular concrete-filled tube columns, *Journal of Constructional Steel Research* 58(12): 1567–1591. http://dx.doi.org/10.1016/S0143-974X(02)00005-6

Eurocode 4 (EC4). 2004. Design of composite steel and concrete structures. Part 1.1: General rules and rules for buildings. London: British Standards Institute, 2004.

Ferretti, E. 2004. On poisson's ratio and volumetric strain in concrete, *International Journal of Fracture* 126(3): 49–55. http://dx.doi.org/10.1023/B:FRAC.0000026587.43467.e6

Gettu, R.; Bazant, Z. P.; Karr, M. E. 1990. Fracture properties and brittleness of high-strength concrete, *ACI Materials Journal* 87(6): 608–618.

Giakoumelis, G.; Lam, D. 2004. Axial capacity of circular concrete-filled tube columns, *Journal of Constructional Steel Research* 60(7): 1049–1068. http://dx.doi.org/10.1016/j.jcsr.2003.10.001

Goldman, A.; Bentur, A. 1993. The influence of microfillers on enhancement of concrete strength, *Cement and Concrete Research* 23(4): 962–972. http://dx.doi.org/10.1016/0008-8846(93)90050-J

Han, L. H.; Ren, Q. X.; Li W. 2011. Testson stub stailess steel-concrete-carbon steel double-skin tubular (DST) columns, *Journal of Constructional Steel Research* 67(3): 437–452.

Han, L. H.; Tao, Z.; Huang, H.; Zhao, X. L. 2004. Concrete-filled double skin (SHS outer and CHS inner) steel tubular beam-columns, *Thin-Walled Structures* 42(9): 1329–1355. http://dx.doi.org/10.1016/j.tws.2004.03.017

Haque, M.; Kayali, O. 1998. Properties of high-strength concrete using a fine fly ash, *Cement and Concrete Research* 28(10): 1445–1452. http://dx.doi.org/10.1016/S0008-8846(98)00125-2

Hatzigeorgiou, G. D. 2008. Numerical model for the behavior and capacity of circular CFT columns, Part I: theory, *Engineering Structures* 30(6): 1573–1578. http://dx.doi.org/10.1016/j.engstruct.2007.11.001

Ho, J. C. M.; Luo, L. 2012. Uni-axial behaviour of normal-strength concrete-filled-steel-tube columns with external confinement, *Earthquake and Structures* 3(6): 889–910. http://dx.doi.org/10.12989/eas.2012.3.6.889

Ho, J. C. M.; Zhou, K. J. H. 2011. Limited deformability design of high-strength concrete beams in low to moderate seismicity regions, *Journal of Civil Engineering and Management* 17(3): 409–423. http://dx.doi.org/10.3846/13923730.2011.594219

Hu, H. T.; Su, F. C. 2011. Nonlinear analysis of short concrete-filled double skin tube columns subjected to axial compressive forces, *Marine Structures* 24(4): 319–337. http://dx.doi.org/10.1016/j.marstruc.2011.05.001

Huang, C.; Yeh, Y. K.; Liu, G. Y.; Hu, H. T.; Tsai, K.; Weng, Y. T.; Wang, S. H.; Wu, M. H. 2002. Axial load behavior of stiffened concrete-filled steel columns, *Journal of Structural Engineering* 128(9): 1222–1230. http://dx.doi.org/10.1016/j.jcsr.2009.09.014

Huang, H.; Han, L. H.; Tao, Z.; Zhao, X. L. 2010. Analytical behaviour of concrete-filled double skin steel tubular (CFDST) stub columns, *Journal of Constructional Steel Research* 66(4): 542–555.

Huang, Y. H.; Wang, R. H.; Huang, X. F. 2011. Calculation of the interfacial tensile stress of CFST members under axial pressure, *Advanced Materials Research* 250–253: 1638–1645.

Kitada, T. 1998. Ultimate strength and ductility of state-of-the-art concrete-filled steel bridge piers in Japan, *Engineering Structures* 20(4): 347–354. http://dx.doi.org/10.1016/S0141-0296(97)00026-6

Köster, W.; Franz, H. 1961. Poisson's ratio for metals and alloys, *Metallurgical Reviews* 6(1): 1–56. http://dx.doi.org/10.1179/mtlr.1961.6.1.1

Kuranovas, A.; Goode, D.; Kvedaras, A. K.; Zhong, S. 2009. Load-bearing capacity of concrete-filled steel columns, *Journal of Civil Engineering and Management* 15(1): 21–33. http://dx.doi.org/10.3846/1392-3730.2009.15.21-33

Lai, M. H.; Ho, J. C. M. 2014. Behaviour of uni-axially loaded concrete-filled-steel-tube columns confined by external rings, *The Structural Design of Tall and Special Buildings* 23(6): 403–426. http://dx.doi.org/10.1002/tal.1046

Lin, M.; Tsai, K. C. 2001. Behavior of double-skinned composite steel tubular columns subjected to combined axial and flexural loads, in *Proc. of the First International Conference on Steel and Composite Structures,* 14–16 June 2001, Pusan, Korea, 1145–1152.

Lu, H.; Han, L. H.; Zhao, X. L. 2010. Fire performance of self-consolidating concrete filled double skin steel tubular columns: experiments, *Fire Safety Journal* 45(2): 106–115. http://dx.doi.org/10.1016/j.firesaf.2009.12.001

Lu, H.; Han, L. H.; Zhao, X. L. 2009. Analytical behavior of circular concrete-filled thin-walled steel tubes subjected to bending, *Thin-Walled Structures* 47(3): 346–358. http://dx.doi.org/10.1016/j.tws.2008.07.004

Lu, X.; Hsu, C. T. T. 2007. Tangent Poisson's ratio of high-strength concrete in triaxial compression, *Magazine of Concrete Research* 59(1): 69–77. http://dx.doi.org/10.1680/macr.2007.59.1.69

Mander, J.; Priestley, M. J. N. 1988. Theoretical stress-strain model for confined concrete, *Journal of Structural Engineering* 114(8): 1804–1825. http://dx.doi.org/10.1061/(ASCE)0733-9445(1988)114:8(1804)

Marzouk, H.; Chen, Z. 1995. Fracture energy and tension properties of high-strength concrete, *Journal of Materials in Civil Engineering* 7(2): 108–116. http://dx.doi.org/10.1061/(ASCE)0899-1561(1995)7:2(108)

Montejo, L. A.; González-Román, L. A.; Kowalsky, M. J. 2012. Seismic performance evaluation of reinforced concrete-filled steel tube pile/column bridge bents, *Journal of Earthquake Engineering* 16(3): 401–424. http://dx.doi.org/10.1080/13632469.2011.614678

O'Shea, M. D.; Bridge, R. Q. 1997. Local buckling of thin-walled circular steel sections with or without internal restraint, *Journal of Constructional Steel Research* 41(2): 137–157. http://dx.doi.org/10.1016/S0143-974X(97)80891-7

Persson, B. 1999. Poisson's ratio of high-performance concrete, *Cement and Concrete Research* 29(10): 1647–1653. http://dx.doi.org/10.1016/S0008-8846(99)00159-3

Sakino, K.; Nakahara, H.; Morino, S.; Nishiyama, I. 2004. Behavior of centrally loaded concrete-filled steel-tube short columns, *Journal of Structural Engineering* 130(2): 180–188. http://dx.doi.org/10.1061/(ASCE)0733-9445(2004)130:2(180)

Schneider, S. P. 1998. Axially loaded concrete-filled steel tubes, *Journal of Structural Engineering* 124(10): 1125–1138. http://dx.doi.org/10.1061/(ASCE)0733-9445(1998)124:10(1125)

Shakir-Khalil, H. 1991. Composite columns of double-skinned shells, *Journal of Constructional Steel Research* 19(2): 133–152. http://dx.doi.org/10.1016/0143-974X(91)90038-3

Szmigiera, E.; Zoltowski, W.; Siennicki, M. 2010. Research on load capacity of concrete filled columns with battened steel sections, *Journal of Civil Engineering and Management* 16(3): 313–319. http://dx.doi.org/10.3846/jcem.2010.36

Tan, K. H.; Zhang, Y. F. 2010. Compressive stiffness and strength of concrete filled double skin (CHS inner & CHS outer) tubes, *International Journal of Mechanics and Materials in Design* 6(3): 283–291. http://dx.doi.org/10.1007/s10999-010-9138-y

Tao, Z.; Han, L. H. 2006. Behaviour of concrete-filled double skin rectangular steel tubular beam–columns, *Journal of Constructional Steel Research* 62(7): 631–646. http://dx.doi.org/10.1016/j.jcsr.2005.11.008

Tao, Z.; Han, L. H.; Wang, D. Y. 2007. Experimental behaviour of concrete-filled stiffened thin-walled steel tubular columns, *Thin-Walled Structures* 45(5): 517–527. http://dx.doi.org/10.1016/j.tws.2007.04.003

Tao, Z.; Han, L. H.; Wang, Z. B. 2005. Experimental behaviour of stiffened concrete-filled thin-walled hollow steel structural (HSS) stub columns, *Journal of Constructional Steel Research* 61(7): 962–983. http://dx.doi.org/10.1016/j.jcsr.2004.12.003

Tao, Z.; Han, L. H.; Zhao, X. L. 2004. Behaviour of concrete-filled double skin (CHS inner and CHS outer) steel tubular stub columns and beam-columns, *Journal of Constructional Steel Research* 60(8): 1129–1158. http://dx.doi.org/10.1016/j.jcsr.2003.11.008

Uenaka, K.; Kitoh, H.; Sonoda, K. 2010. Concrete filled double skin circular stub columns under compression, *Thin-Walled Structures* 48(1): 19–24. http://dx.doi.org/10.1016/j.tws.2009.08.001

Varma, A. H.; Ricles, J. M.; Sause, R.; Lu, L. W. 2002. Seismic behavior and modeling of high-strength composite concrete-filled steel tube (CFT) beam–columns, *Journal of Constructional Steel Research* 58(5): 725–758. http://dx.doi.org/10.1016/S0143-974X(01)00099-2

Wei, S.; Mau, S. T.; Vipulanandan, C.; Mantrala, S. K. 1995. Performance of new sandwich tube under axial loading: experiment, *Journal of Structural Engineering* 121(12): 1806–1814. http://dx.doi.org/10.1061/(ASCE)0733-9445(1995)121:12(1806)

Wright, H.; Oduyemi, T.; Evans, H. 1991. The experimental behaviour of double skin composite elements, *Journal of Constructional Steel Research* 19(2): 97–110. http://dx.doi.org/10.1016/0143-974X(91)90036-Z

Yang, J.; Xu, H.; Peng, G. 2008. Behavior of concrete-filled double skin steel tubular columns with octagon section under axial compression, *Frontiers of Architecture and Civil Engineering in China* 2(3): 205–210. http://dx.doi.org/10.1007/s11709-008-0035-5

Yang, Y.; Han, L. H. 2008. Concrete-filled double-skin tubular columns under fire, *Magazine of Concrete Research* 60(3): 211–222. http://dx.doi.org/10.1680/macr.2007.00074

Young, B.; Ellobody, E. 2006. Experimental investigation of concrete-filled cold-formed high strength stainless steel tube columns, *Journal of Constructional Steel Research* 62(5): 484–492. http://dx.doi.org/10.1016/j.jcsr.2005.08.004

Zhao, X. L.; Grzebieta, R. 2002. Strength and ductility of concrete filled double skin (SHS inner and SHS outer) tubes, *Thin-Walled Structures* 40(2): 199–213. http://dx.doi.org/10.1016/S0263-8231(01)00060-X

Zhao, X. L.; Grzebieta, R.; Elchalakani, M. 2002. Tests of concrete-filled double skin CHS composite stub columns, *Steel and Composite Structures* 2(2): 129–146. http://dx.doi.org/10.12989/scs.2002.2.2.129

Zhao, X. L.; Tong, L. W.; Wang, X. Y. 2010. CFDST stub columns subjected to large deformation axial loading, *Engineering Structures* 32(3): 692–703. http://dx.doi.org/10.1016/j.engstruct.2009.11.015

Zhou, F.; Barr, B.; Lydon, F. 1995. Fracture properties of high strength concrete with varying silica fume content and aggregates, *Cement and Concrete Research* 25(3): 543–552. http://dx.doi.org/10.1016/0008-8846(95)00043-C

List of Notations

NSC – Normal-strength concrete;
HSC – High-strength concrete;
RC – Reinforced concrete;
HSCFST – High-strength concrete-filled-steel-tubular;
D_i – Outer diameter of inner tube of double-skinned CFST column;
D_o – Outer diameter of outer tube of double-skinned CFST column;
t_i – Thickness of inner tube of double-skinned CFST column;
t_o – Thickness of outer tube of double-skinned CFST column;
A_i – Cross-section area of the inner tube;
A_o – Cross-section area of the outer tube;
A_{cc} – Cross-section area of the core concrete;
A_R – Cross-section area of external steel ring;
c – Hollow section ratio;
N – Load-carrying capacity of a specimen;
N_i – Axial load taken by the inner tube;
N_o – Axial load taken by the outer tube;
N_{cc} – Axial load taken by the core concrete;
N_{EC} – Axial strength of a specimen evaluated by Eurocode;
N_p – Axial strength of a specimen predicted by the proposed model;
N_p' – Axial strength of a specimen predicted by simplified formula;
N_{p1} – Axial strength of a specimen predicted by Tan and Zhang (2010);
N_{p2} – Axial strength of a specimen predicted by Tao *et al.* (2004);
N_{p3} – Axial strength of a specimen predicted by Uenaka *et al.* (2010);
N_{p4} – Axial strength of a specimen predicted by Hu and Su (2011);
N_t – Experimentally measured axial strength of a specimen;
f_i – Axial stress of the inner tube;
f_o – Axial stress of the outer tube;
f_{cc} – Axial stress of the core concrete;
f_c' – Uni-axial concrete compressive strength represented by cylinder strength;
f_r – Confining pressure;
ε_y – Axial strain at yielding;
ε_{lb} – Axial strain for local buckling;
E_{so} – Elastic modulus of outer steel tube;
$\sigma_{\theta,i}$ – Hoop stress in inner tube of double-skinned CFST column;
$\sigma_{\theta,o}$ – Hoop stress in outer tube of double-skinned CFST column;
σ_R – Tensile stress in external steel ring;
$f_{y,i}$ – Uni-axial yield strength of inner tube of double-skinned CFST column;
$f_{y,o}$ – Uni-axial yield strength of outer tube of double-skinned CFST column;
$f_{y,R}$ – Uni-axial yield strength of external steel ring;
n – Number of external steel bars;
h – Height of the specimen;
s – Spacing of external steel rings;
α_θ – Hoop stress ratio.

Chun Xiao DONG. (MPhil) is a PhD student of the Department of Civil Engineering of The University of Hong Kong. His research deals with the uni-axial behaviour of double-skinned concrete-filled-steel-tubular column with external confinement as well as its respective theoretical model.

Johnny Ching Ming HO. (PhD, MHKIE, MIEAust, CPEng, MIStructE, CEng) is a senior lecturer in the School of Civil Engineering, The University of Queensland. His research interests are on developing scientific mix design for high-performance concrete using multi-sized fillers and its application on concrete-filled-steel-tubular columns.