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Uni-axial behaviour of normal-strength concrete-filled-steel-tube columns with external confinement

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Abstract. Because of the heavy demand of confining steel to restore the column ductility in seismic regions, it is more efficient to confine these columns by hollow steel tube to form concrete-filled-steel-tube (CFST) column. Compared with transverse reinforcing steel, steel tube provides a stronger and more uniform confining pressure to the concrete core, and reduces the steel congestion problem for better concrete placing quality. However, a major shortcoming of CFST columns is the imperfect steel-concrete interface bonding occurred at the elastic stage as steel dilates more than concrete in compression. This adversely affects the confining effect and decrease the elastic modulus. To resolve the problem, it is proposed in this study to use external steel confinement in the forms of rings and ties to restrict the dilation of steel tube. For verification, a series of uni-axial compression test was performed on some CFST columns with external steel rings and ties. From the results, it was found that: (1) Both rings and ties improved the stiffness of the CFST columns and (2) the rings improve significantly the axial strength of the CFST columns while the ties did not improve the axial strength. Lastly, a theoretical model for predicting the axial strength of confined CFST columns will be developed.

Keywords: columns; concrete-filled-steel-tube; external confinement; normal-strength concrete; rings; ties

1. Introduction

It is commonly accepted that the strength, ductility and deformability performance of reinforced concrete (RC) columns can be significantly improved by installing more confining steel (Li *et al*. 1991, Watson and Park 1996, Bayrak and Sheikh 1998, Paultre *et al*. 2001, Ho and Pam 2003, Sullivan 2010, Zhou *et al*. 2010, Ho 2011a) within the potential plastic hinge or critical regions (Pam and Ho 2009, Yan and Au 2010, Ho 2011b), by steel-concrete composite columns (Young and Ellobody 2006, Tokgoz and Dundar 2008, Uenaka *et al*. 2008, Gonçalves and Camotim 2010, Valente and Cruz 2010, Zhao and Yuan 2010), or by FRP-concrete composite columns (Lam and Teng 2009, El-Shihy *et al*. 2010, Wu and Wang 2010, Wu and Wei 2010). Nevertheless, the effectiveness of strength and ductility improvement due to addition of confinement steel decreases as the concrete strength increases (Pam and Ho 2001, Lam *et al*. 2009a, Ho *et al*. 2010). Therefore, if the same level of strength and ductility/deformability provided previously to columns of lower concrete strength are to be maintained, the content of confining steel required for RC columns of higher concrete strength will increase dramatically (Bayrak and Sheikh 1998, Paultre *et al*. 2001,

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Lam *et al*. 2009b). Consequently, it causes problematic steel congestion problem within the plastic hinge region of columns, or in the proximity of beam-column joints, that adversely affects the quality of concrete placing at these locations. Possible problems at these locations include segregation and plastic settlement crack of concrete.

To resolve the problems and maintain sufficient confinement for ductility provision, composite concrete-filled-steel-tube (CFST) columns are advocated. As per various theoretical and experimental research performed by researchers in the past few decades, it was found that CFST columns had superior performance to ordinary RC columns under uni-axial load (Furlong 1967, Knowles and Park 1969, Usami and Fukumoto 1984, Sasaki 1984, Bradford 1996, Ellobody and Young 2006, Huang *et al*. 2008, Dai and Lam 2010), under flexure (Usami and Ge 1994, Nakanishi *et al.* 1999, Han 2002, Han *et al*. 2006, Choi *et al*. 2007, Lee 2007, Fang and Zhu 2009, Hu *et al*. 2010, Park *et al*. 2010) and under torsion (Hsu *et al*. 2009). From safety point of view, they are characterised by higher strength, ductility, buckling resistance and energy absorption before failure, which give ample warning against collapse during earthquake attack. From cost effectiveness point of view, the tubes act as both the longitudinal reinforcement and formwork that save the time and cost for steel bars fixing and formwork fabrication. Hence, the construction cycle is reduced considerably. From environmental point of view, the size of CFST columns could be 10 to 30% smaller than that of ordinary RC columns due to steel and concrete composite action. Hence, they utilise less cement and concrete, which reduce the construction and demolition waste, as well as the embodied energy and carbon levels of the building structures. All of these help produce a more sustainable construction environment. Lastly, the floor area saved is always beneficial to the developers, architects and engineers.

Notwithstanding the numerous benefits, one of the major shortcomings of the CFST columns is that the steel tube is not fully effective in confining the core concrete during initial elastic stage when the strain in concrete is still small (Kitada 1998). Under such a circumstance, the steel tube dilates more than concrete and thereby causing imperfect steel-concrete interface bonding (Uy 2001, Elremaily and Azizinamini 2002, Giakoumelis and Lam 2004, Sakino *et al*. 2004, Yin and Lu 2010). The imperfect bonding in turns adversely affect the confining pressure provide to the concrete core, stiffness and axial deformation of the CFST columns (Roeder *et al.* 1999, Johansson and Gylltoft 2002, Nezamen *et al.* 2006, Elchalakani and Zhao 2008, Aly *et al.* 2010). Moreover, due to the imperfect interface bonding, the confining pressure provided by the steel tube is not uniform at the level of confinement even for column of circular cross-section. It is only when the strain in concrete has increased to roughly the ultimate concrete strain that the confining pressure provided by the steel tube will increase substantially because of the increased Poisson's ratio and dilation of concrete core (Persson 1999, Ferretti 2004, Lu and Hsu 2007).

To improve the confining effectiveness of the steel tube, it is proposed in this study to use external steel confinement to restrict the dilation of the steel tube under compression. The external steel confinement can normally be adopted in the construction of new structural members instead of application to retrofit existing structures. The proposed external confinement is divided into two types, which are in the form of steel rings welded around the outer perimeter of the CFST columns, and steel ties installed in the plane of column cross-section tightened by nuts against the external face of steel tube. It should be noted that both ends of the nuts are installed just tight, which means there is no initial pre-stressing force applied to the steel ties. Since these forms of external confinement will not be subjected to axial compression even the CFST columns do, they act as passive confinement to confine both the steel tube and concrete core by restraining their lateral dilations. Previously, similar efforts have been adopted by some researchers to improve the steel-concrete interface bonding. Nevertheless, instead of external confinement, internal steel ties (Hu *et al*. 2005), shear studs (Nakanishi *et al*. 1999) or welded steel stiffeners (Wright 1995, Huang *et al*. 2002) were adopted to improve the interface bonding. Compared with the proposed form of external confinement, the internal shear studs and stiffeners are more difficult to be installed in columns of building structures.

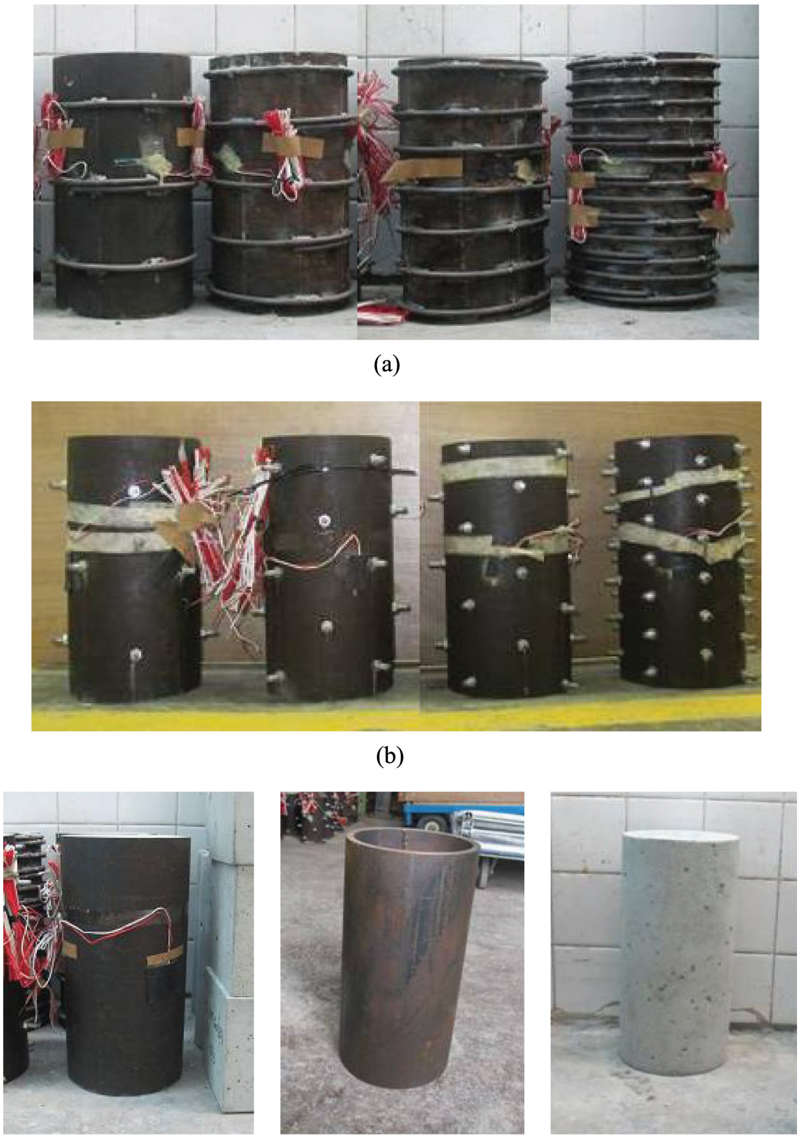
In order to verify the effectiveness of the proposed external confinement in restricting the lateral dilation of CFST columns, the authors have in this study carried out a series of uni-axial compression test for CFST columns confined by external steel rings and ties. The obtained load-displacement curves, Poisson's ratio, axial strength and stiffness for these columns were then compared with those of CFST columns without external confinement. Uni-axial compression tests for standard concrete cylinder (150 mm diameter) and hollow steel tube have also been conducted. From the test results, it is evident that: (1) The external steel rings confinement could improve significantly the strength and elastic stiffness of CFST columns. (2) The external steel ties confinement could improve the stiffness of CFST columns, but not the axial strength. (3) The axial strain of all CFST columns, with or without external confinement, can reach at least 25% without significant degradation of axial strength, which is much more superior to those of plain concrete cylinder and bare hollow steel tube.

2. Test programme

2.1 Test setup and details of specimens

In this study, a total of 9 CFST column specimens have been fabricated and tested in a uni-axial compression machine of capacity 5,000 kN. The CFST column specimens are divided into 3 groups: (1) Four CFST columns with external steel rings at different spacing ($5t$, $10t$, $15t$ and $20t$, where t is the thickness of the steel tube) cast with normal-strength concrete, (2) Four CFST columns with external steel ties at different spacing ($5t$, $10t$, $15t$ and $20t$) cast with normal-concrete strength and (3) One CFST columns without external confinement cast with normal-strength concrete for comparison purpose. The normal-strength concrete had concrete cylinder strength of about 30 MPa on testing day. The grade of steel is *S*355 produced as per BS EN 10210-2:2006 and with measured yield strength of 380 MPa. The thickness of the steel tube t is 5 mm for all CFST specimens. The diameter of the CFST columns measured to the outer face of steel tube is 168 mm. The height of the CFST columns is 330 mm, which gives an aspect ratio of about 2. Fig. 1(a) shows the photo of the CFST column specimens with various spacing of steel rings as external confinement. Fig. 1(b) shows the photo of CFST column specimens with various spacing of steel ties as external confinement. Fig. 1(c) shows the photo of the CFST column specimen without external confinement. Fig. 1(d) shows the photo of the bare hollow steel tube and Fig. 1(e) shows the photo of the standard concrete cylinder of diameter 150 mm.

All the external confinement was provided at spacing of $5t$, $10t$, $15t$ and $20t$, which are 25, 50, 75 and 100 mm respectively. For the external steel rings, the diameters of all ring adopted are 8 mm and the nominal and actual yield strength is 250 and 357 MPa respectively. The steel rings were welded to the steel tube at 8 locations in each level of confinement, which were separated from each other by an angle of 45° at the centre of cross section. At the end of the steel rings, there was



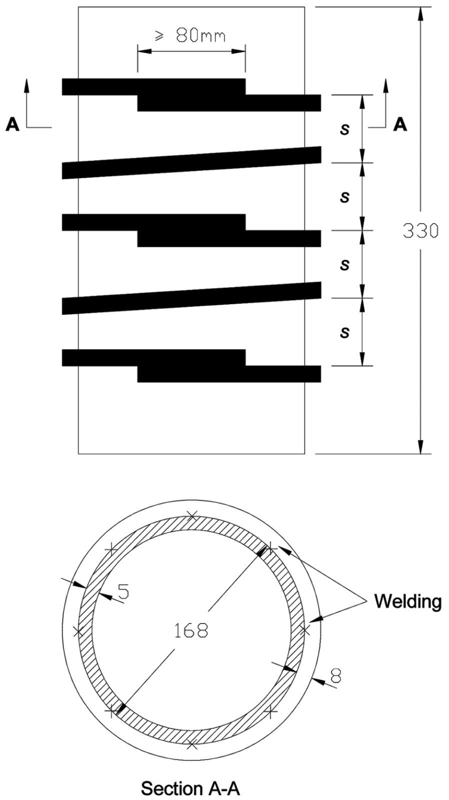
(a)

(b)

(c) (d) (e)

Fig. 1 Photos of the test specimens: (a) CFST columns with steel rings ($s = 20t$, $15t$, $10t$ and $5t$), (b) CFST columns with steel ties (s=$20t$, $15t$, $10t$ and $5t$), (c) CFST column without external confinement, (d) hollow steel tube and (e) plain concrete cylinder

an overlapping length of 10 times the steel diameter, which is 80 mm. This is to ensure that sufficient anchorage length was provided to develop its yield strength of the steel rings. Fig. 2 shows the details of the steel rings (welding locations and overlapping length) and the respective CFST specimens. For the steel ties, the diameter is 8 mm and with nominal and actual yield strength of 250 and 357 MPa respectively. The ties adopted in this test were all threaded. Also, they were fabricated to be slightly longer than the outer diameter of the steel tube so that nuts could be installed at both of its ends to tighten them against the external face of the steel tube. The nuts were



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

Fig. 2 Details of arrangement of steel rings

installed just tight and therefore no initial pre-stressing force was applied to the steel tube. At each level, a pair of steel ties (with a level difference of a steel diameter) was installed perpendicular to each other. The pair of ties was then rotated by 45° at the next level and the arrangement continued for the subsequent layers of ties. There was 1 mm tolerance for the hole drilled on the hollow steel tube for accommodating the ties. The details of the steel tie arrangement and the respective CFST specimens are illustrated in Fig. 3.

Each of the CFST column specimens was given a unique code. For example, 30-5-5R represents a CFST column specimen with concrete cylinder strength of about 30 MPa (indicated by the first number), thickness of steel tube t equal to 5 mm (indicated by the second number), spacing of external confinement equal to $5t$ (indicated by the last number) and the suffix R stands for rings as external confinement, where T stands for ties as external confinement. The CFST column without external confinement is represented by 30-5-00X, where X stands for no confinement.

Apart from CFST column specimens, a 150 mm diameter plain concrete cylinder and a hollow steel tube were tested under uni-axial compression. The hollow steel tube is represented by H-5, where H stands for hollow steel tube and 5 for the thickness in mm. The plain concrete cylinder is represented by C-30, where C stands for concrete cylinder and 30 for concrete strength in MPa. Table 1 summarises the cross-section properties and details of external confinement, if any, for each of the tested CFST column specimens. All the tests were carried out using a uni-axial compression testing machine of safe working capacity of 5,000 kN as shown in Fig. 4.

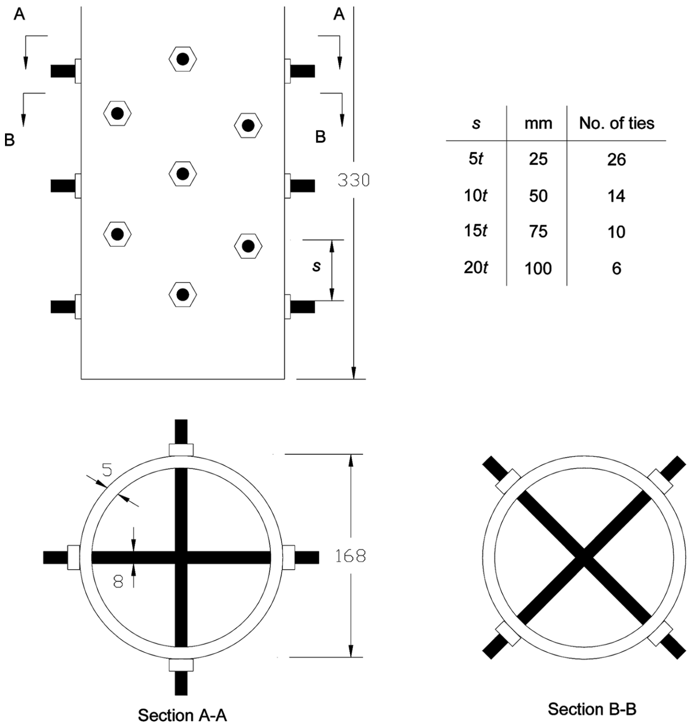


Fig. 3 Details of arrangement of steel ties

Table 1 Cross-section properties and details of external confinement

External confinement	Specimen code	Sectional dimensions			Concrete strength	Spacing	Confinement	
		D (mm)	t (mm)	H (mm)	f_c' (MPa)	s (mm)	D_c (mm)	f_y (MPa)
Ring	30-5-5*R*	168	5	330	30	25	8	250
	30-5-10*R*					50		
	30-5-15*R*					75		
	30-5-20*R*					100		
Tie	30-5-5*T*	168	5	330	30	25	8	250
	30-5-10*T*					50		
	30-5-15*T*					75		
	30-5-20*T*					100		
None	30-5-00*X*	168	5	330	30	0	0	0

Notes:
D = Diameter of CFST column section
t = Thickness of steel tube
H = Length of CFST column
f_c' = Uni-axial concrete compressive strength represented by cylinder strength
s = Spacing of external confinement
D_c = Diameter of external confinement section
f_y = Yield strength of steel bar

(a) (b)

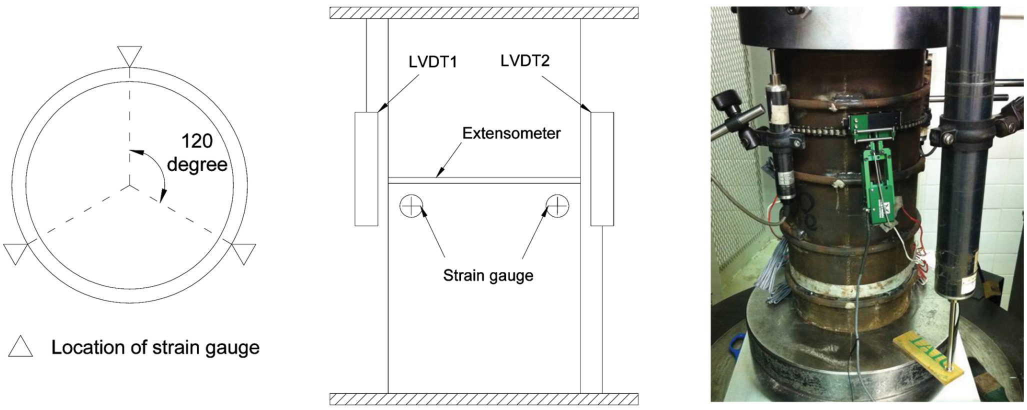
Fig. 4 Test set-up: (a) 5,000 kN uni-axial compression machine and (b) set up of test specimen inside the machine

2.2 Instrumentation

The following types of instrumentation were used for monitoring the behaviour of column specimens:

(1) Strain gauges – Two directional strain gauges, which could measure the longitudinal and transverse strains, were installed on the steel tube of each of the CFST columns and on the hollow steel tube. Three strain gauges of this type were installed in every specimen, which were equally spaced with an angle of 120° at the centre of cross section. The arrangement of strain gauges is shown in Figs. 5(a) and (b).

(2) Linear variable differential transducer (LVDT) – Two LVDTs were used in the test to measure the axial deformation of the specimens under uni-axial compressive load. An LVDT was installed to



(a) (b) (c)

Fig. 5 Arrangement of strain gauges, LVDTs and extensometer: (a) arrangement of strain gauges, (b) arrangement of LVDTs and extensometer and (c) photo of instrumentation measure the upward movement of the bottom platen due to the applied axial load. As the top steel platen would also move upward due to the applied axial load, another LVDT was installed to measure the upward movement of the top platen during load application. The actual axial shortening of the test specimens can be obtained by the difference of the readings measured by both LVDTs. The arrangement of the LVDTs is also shown in Figs. 5(b) and (c).

(3) Circumferential extensometers – A circumferential extensometer of maximum measuring range of 6 mm was installed around the centre of each of the specimens to measure the dilation of the column at the initial stage of the loading process. In the CFST column specimens and the hollow steel tube specimen, the extensometer was removed when the measured lateral dilation of specimens was about to reach the maximum measuring range of 6 mm. For plain concrete cylinder, the extensometer was removed when the longitudinal strain has reached slightly above 0.002. The arrangement of the extensometer is also shown in Figs. 5(b) and (c).

2.3 Testing procedure

All the loading tests were displacement-controlled. For CFST column and hollow steel tube specimens, the loading was applied with a displacement rate of 0.3 mm/min. This rate would be increased incrementally by 0.2 mm/min for every 10 mm increase in axial displacement after the specimens had reached the yielding stage, where the axial displacement increased significantly with relatively constant applied load. For plain concrete cylinder, the loading rate was kept at 0.3 mm/min unchanged. The loading test will be stopped either when the applied load has dropped to less than 80% of the maximum measured load in the post-peak range, or when the axial displacement had reached about 80 mm (corresponding to 25% axial strain), whichever was earlier.

3. Experimental results

3.1 Axial load against axial shortening curves

Fig. 6 shows the axial load against axial shortening curves of CFST column specimens with spacing of external confinement (rings and ties) equal to $10t$, CFST column specimens without external confinement, hollow steel tube and plain concrete cylinder. The value in the y-axis represents the total axial load applied to the specimens by the loading machine, whereas that the x-axis represents the axial shortening of the specimens under axial load. The total load applied is directly obtained from the output of the compression machine, whereas the axial shortening of the CFST specimens is obtained by the difference between the displacement of the movement of the top and bottom loading platens measured by two different LVDTs. From the figure, it can be observed that the CFST specimens (with and without external confinement) have superior uni-axial performance in terms of strength, stiffness and deformability to that of bare hollow steel tube and plain concrete cylinder.

Amongst the CFST columns, specimens with external confinement performed better in terms of strength and stiffness than the CFST columns without external confinement. This can be verified by the obtained axial load-shortening curves of 30-5-10*R*, 30-5-10*T* and 30-5-00*X* as shown in Fig. 6. The reasons are that the external confinement, i.e., steel rings and ties, effectively restricted the lateral dilation of the steel tube under uni-axial compression. Therefore, larger confining pressure

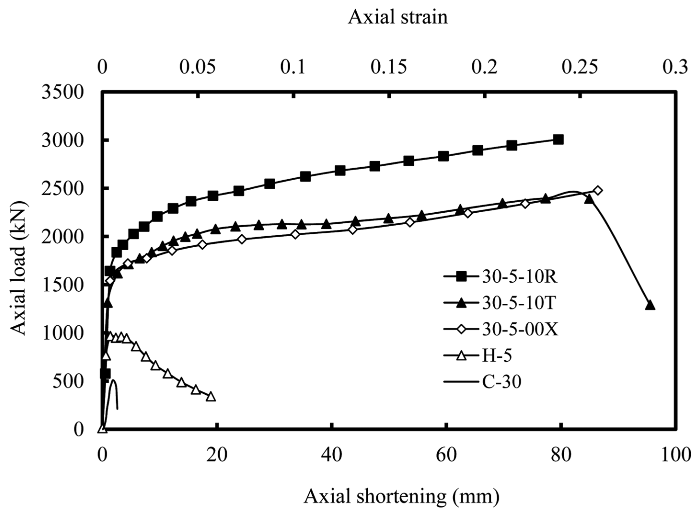


Fig. 6 Axial load-shortening curves of CFST column specimens, hollow steel tube and plain concrete cylinder

was provided to confine the steel tube as well as the concrete core. The strength, stiffness and deformability were therefore enhanced. For the CFST column without external confinement, the steel tube dilated more under the same axial load because the Poisson's ratio of steel is larger than that of concrete. It would then adversely affect the steel-concrete interface bonding and decrease the confining pressure provided by the steel tube.

On the other hand, between the two types of external confinement, the steel rings performed better than the steel ties provided at the same spacing in terms of strength, stiffness and deformability. This is because the steel rings were able to provide a more uniform confining pressure to both the steel tube and the concrete core around the circumference of the circular steel tube, although there would be some variations of confining pressure between the steel rings in longitudinal direction. Compared with the steel rings, the steel ties could only provide the strongest confining pressure at the locations where the ties were tightened against the steel tube. Between these locations in the planes of both horizontal and vertical directions, the confining pressure decreases. Furthermore, the openings (holes) for installation of steel ties form the weak points in the steel tube under uni-axial load because of the reduced moment of inertia. At large longitudinal axial strains, horizontal splitting stress would be concentrated at the openings and caused longitudinal crack to form along the steel tube, which started at the openings at roughly the mid-height of the steel tube and between the steel ties (i.e., furthest away from the points of effective lateral restraint). The failure modes of both CFST columns with different forms of external confinement as well as that of the column without external confinement will be discussed in the later section.

For CFST columns confined by external steel rings, the stiffness and axial strength increased as the spacing of rings decreased. Moreover, the strength and stiffness were always larger than those of the CFST columns without external confinement. This can be seen by the axial load-shortening curves plotted in Figs. 7(a) and (c) for all ring-confined CFST columns with various spacing. When the ring spacing decreased, a more uniform confining pressure was provided to the CFST specimens in the longitudinal direction. Thus, the strength and stiffness of the rings were enhanced due to larger confining pressure. However, for CSFT columns confined by external steel ties, it can be seen from their axial load-shortening curves in Fig. 7(b) that the strength were not enhanced (see also

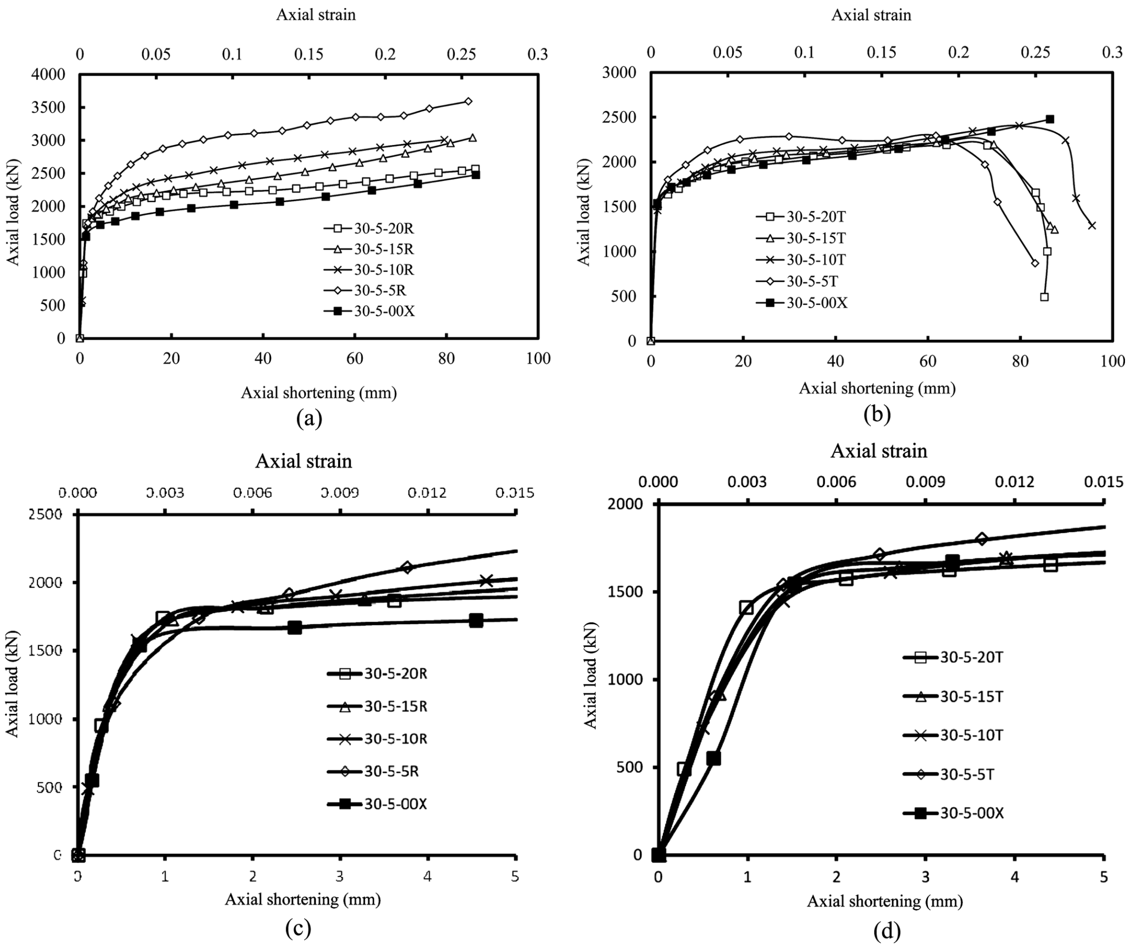
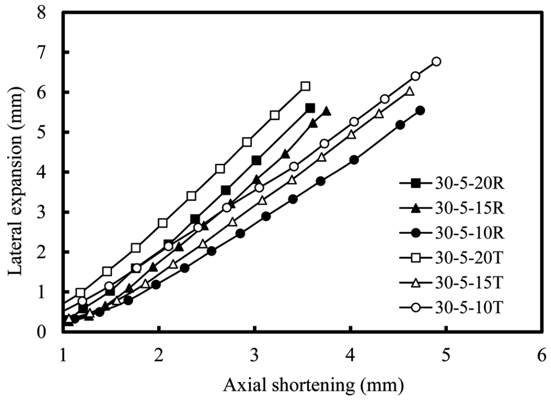


Fig. 7 Axial load-shortening curves of CFST columns with external confinement: (a) ring-confined CFST columns, (b) tie-confined CFST columns, (c) ring-confined CFST columns (initial portion) and (d) tie-confined CFST columns (initial portion)

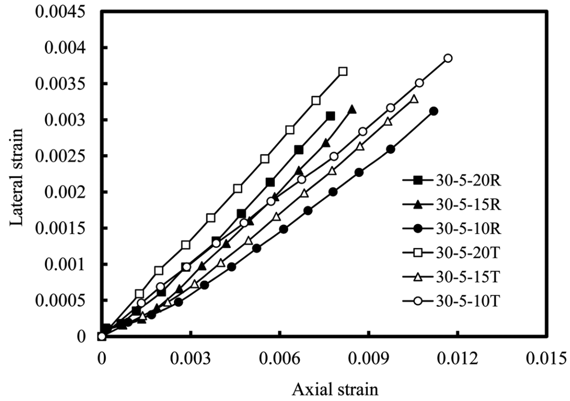
Table 3). Therefore, the ties were not effective for axial strength improvement. However, the stiffness of columns with all ties spacing were larger than those of the columns without external confinement (see also Table 3 and Fig. 7(d)). Also, the strength and stiffness increased as the tie spacing reduced (except for axial strength of 30-5-5*T*, which will be explained later). This is because when the steel ties were provided at large spacing, longitudinal cracks forms at early stage which started at the tie openings due to relatively small confining pressure. The cracks then propagated along the height of the columns in both directions and destroyed the effective restraints originally provided by the steel ties. Therefore, the effective length of the CFST columns increased, and hence prompted to early buckling of the steel tube.

3.2 Lateral strain against longitudinal strain curves

It has been pointed out by previous researchers (Kitada 1998, Uy 2001, Elremaily and



(a)



(b)

Fig. 8 Measured lateral expansion and lateral strain of CFST columns: (a) lateral expansion against axial shortening for CFST columns and (b) lateral strain against axial strain for CFST columns

Azizinamini 2002, Giakoumelis and Lam 2004, Sakino *et al.* 2004, Yin and Lu 2010, Liao *et al.* 2011) that because of the different Poisson's ratios of steel (~0.3) and concrete (~0.2), the steel tube is not fully effective to confine the concrete core during initial elastic stage under axial compression. Therefore, it is proposed in this study to use external confinement to restrict the lateral dilation of the steel tube under axial compression and hence improve the steel-concrete interface bonding and the confining pressure provided to the concrete core. At the same time, the external confinement also provides some confining pressure to the steel tube to improve the axial strength of the steel tube and hence the capacity of the CFST columns. The effectiveness of the external confinement could be studied by plotting the lateral expansion (dilation) measured at the mid-height of the steel tube by the extensometer against the axial shortening of the CFST specimens during the initial elastic stage.

Fig. 8(a) shows the lateral expansion against axial shortening for the CFST columns with external confinement of various spacings. Fig. 8(b) shows the respective graphs of lateral strain against axial strain for the CFST columns. The lateral strain is obtained by dividing the circumferential expansion by the circumference of the steel tube. The axial strain is obtained by dividing the axial shortening of the CFST columns by the overall height of the specimen. The Poisson's ratio of the CFST columns is calculated from the initial slope of the graph before reaching 1/3 of the maximum load. The results are summarised in Table 2 for all the CFST columns. From the table, it can be seen that the Poisson's ratios of all ring-confined CFST columns, as well as the tie-confined CFST columns with close spacing (i.e., $10t$ and $15t$), are about or smaller than 0.2, which is regarded as the average Poisson's ratio of normal-strength concrete. Hence, it indicates that both types of external confinement, when provided at suitable spacing, can effectively restrict the lateral dilation of steel tube and maintain an intact steel-concrete interface bonding. It should also be noted that no data of lateral expansion and strain were obtained for CFST columns with $5t$ confinement spacing. This was because the spacing was too small to install the circumferential extensometer. Nevertheless, it can be deduced from the trend that the Poisson's ratio for these CFST columns will be smaller than 0.2.

On the other hand, for CFST columns confined by steel ties, the evaluated Poisson's ratio of column with tie spacing of $20t$ is slightly larger than 0.2, which indicates that the steel ties at such

Table 2 Poisson's ratio of tested CFST specimens

Specimen code	Poisson's ratio
30-5-10*R*	0.172
30-5-15*R*	0.205
30-5-20*R*	0.219
30-5-10*T*	0.201
30-5-15*T*	0.197
30-5-20*T*	0.235

Table 3 Axial strength enhancement ratio and stiffness enhancement ratio

Specimen code	Axial strength (kN)	Axial strength enhancement ratio ω_S	Stiffness (kN/mm)	Stiffness enhancement ratio ω_E
30-5-5*R*	3590	1.450	1341	1.354
30-5-10*R*	3006	1.214	1321	1.334
30-5-15*R*	3041	1.228	1295	1.308
30-5-20*R*	2566	1.036	1265	1.277
Average		1.232		1.319
30-5-5*T*	2329	0.941	1332	1.345
30-5-10*T*	2408	0.973	1233	1.245
30-5-15*T*	2228	0.900	1199	1.211
30-5-20*T*	2199	0.888	1141	1.153
Average		0.925		1.238
30-5-00*X*	2476	1.000	990	1.000
H-5	981	0.396	944	0.954
C-30	514	0.208	306	0.309

large spacing are not capable of effectively restricting the lateral dilation of the steel tube in CFST columns. Therefore, the confining pressure, axial strength and stiffness would be smaller than those of CFST columns confined by steel rings at the same spacing. Nonetheless, when compare with the Poisson's ratio of steel (about 0.3), the Poisson's ratios of this tie-confined CFST column is still smaller. In other words, it can reasonably be deduced that the steel ties were still able to restrict partially the lateral dilation of the steel tube, although the steel-concrete interface bonding may not be perfect. For other set of tie-confined CFST columns with spacing of 10t and 15t, the Poisson's ratio are about 0.2, which indicates that for closely spaced tie-confined CFST columns, the confinement is able to effectively restrict the lateral expansion of the steel tube, and thereby resuming an intact steel-concrete interface bonding. It is also observed that for these columns, the axial strength and stiffness increases as the tie spacing decreases, because of the larger confining pressure provided.

3.3 Failure modes

Figs. 9(a), (b) and (c) show the photos of the failure modes of CFST columns without external

(a)

(b)

(c)

Fig. 9 Photos of CFST columns: (a) CFST column without external confinement (30-5-00*X*), (b) ring-confined CFST columns ($s = 20t$, $15t$, $10t$ and $5t$) and (c) tie-confined CFST columns at failure ($s = 20t$, $15t$, $10t$ and $5t$)

confinement, with ring confinement and tie confinement respectively. From the figures, it can be observed that the failure mode of CFST columns without external confinement is due to the overall buckling of steel tube at large axial strain. Because there was no external confinement, the effective length (i.e., the length between lateral restraints) was equal to the height between the top and bottom loading platen. Hence, buckling was easier to occur when compare with other externally confined CFST columns. For the ring-confined CFST columns, since the rings were able to restrict the lateral expansion of CFST columns under compression, the rings provided effective lateral restraints against buckling at each level of the confinement. The effective length of the rings-confined CFST columns was then reduced to only the ring spacing.

On the other hand, the failure mode of tie-confined CFST columns is slightly complicated. Firstly, because the confining pressure provided for these columns at the level of confinement was not uniform in the plane of cross-section, the ties were not able to provide effective lateral restraint against buckling of the steel tube at each level of the confinement under large axial strain. It then increased the effective length of the CFST columns and buckling was easier to occur. Secondly, the openings for tie installation reduced the second moment of area of the cross section and thus decreased the critical buckling load of the CFST columns. Thirdly, when the specimens were subjected to large axial as well as lateral stains, there was large horizontal splitting stress concentrated at the openings. When the lateral strains of the steel tube at these locations continued to increase, longitudinal crack would start to propagate from the tie opening near the mid-height towards both ends of the steel tube. This is shown in Fig. 9(c) by the large longitudinal steel crack.

3.4 Maximum axial strain at failure

For ring-confined CFST columns, it can be observed from Fig. 7(a) that the specimens were able to reach at least 80 mm of axial shortening (or at least 25% axial strain) without significant degradation in axial strength. This is because of the additional confining pressure provided by the rings to both the steel tube and concrete core that improves their strength and ductility. There is no significant difference in the axial strain for CFST columns with different ring spacing. This is probably because the ultimate axial strain for these columns is larger than the capacity of the machine. Nevertheless, for CFST columns with smaller spacing of rings, the same axial strain can be reached at higher axial strength, which indicates a larger maximum design limit of axial strength and deformability that can be achieved simultaneously.

However, for tie-confined CFST columns, it is observed in Fig. 7(b) that a significant drop of axial strength occurred at an axial strain of about 20%. More importantly, it is seen that the earliest strength degradation happened in 30-5-5*T*, which has a closest spacing of steel ties and the largest confining pressure amongst all tie-confined CFST columns. A possible reason explaining this phenomenon could be the excessively large number of openings drilled on the steel tube for ties installation. Because of the closely-spaced openings, the propagation of longitudinal splitting crack was expedited and led to abrupt axial strength degradation. It therefore implies that a minimum spacing of tie confinement should be limited in the design to avert the rapid strength degradation under large axial strain.

3.5 Improvement on axial strength and stiffness due to external confinement

Table 3 summarises the axial strength of the CFST columns, hollow steel tube and plain concrete cylinders. It can be seen from the table that the CFST columns have axial strength much higher than the bare hollow steel tube as well as plain concrete cylinders. Amongst the CFST columns, the axial strength of CFST columns with external steel rings is larger than that of CFST columns without external confinement. The reason is that for these columns, uniform confining pressure is provided by the steel tube and external steel rings to restrict the dilation of steel tube and core concrete. However, for CFST columns confined by tie, there is no axial strength improvement. This is because the second moment of area of the cross-section has been reduced due to the drilling of tie openings. Moreover, amongst the CFST columns with external steel rings, the strength enhancement increases as the spacing of rings decreases. Nonetheless, for CFST columns with external steel ties, the axial strength enhancement ratio increases as the tie spacing decreases up to $10t$. For tie spacing of $5t$, the axial strength enhancement drops because of the excessive number of tie opening drilled on the steel tube.

The relative strength increase due to the installation of external confinement is studied by the axial strength enhancement ratio ω_S defined in Eq. (1)

$$\omega_S = \frac{P_c}{P} \tag{1}$$

where P_c and P are the experimentally measured axial strengths (in kN) of CFST columns with and without external confinement respectively. The maximum axial strength is taken as the maximum measured value during the testing process. The values of ω_S are listed in Table 3 for all CFST columns. From the table, it is evident that the average axial strength enhancement ratios are about 1.232 and 0.925 for CFST columns confined by steel rings and ties respectively. The maximum value of ω_S is 1.450 for ring-confined CFST columns and 0.973 for tie-confined CFST columns. The larger strength enhancement in rings-confined CFST columns is due to the larger and more uniform confining pressure provided by the steel rings than the steel ties. From the results, it can be seen that external steel rings are more effective than ties to improve the axial strength of CFST columns.

The enhancement in stiffness of the CFST columns is studied by the stiffness enhancement ratio ω_E, which is defined by Eq. (2)

$$\omega_E = \frac{E_c}{E} \tag{2}$$

where E_c and E are the experimentally measured stiffness (in kN/mm) of CFST columns with and without external confinement respectively. The values of ω_E are also listed in Table 3 for all CFST columns. From the table, it is evident that the trend of the value of ω_E follows similarly to that of ω_S because of similar reasons except that the stiffness enhancement of tie-confined CFST columns increases consistently with decreasing tie spacing. The average stiffness enhancement ratios are about 1.319 and 1.238 for CFST columns confined by steel rings and ties respectively. The maximum value of ω_E is 1.354 for ring-confined CFST columns and 1.345 for tie-confined CFST columns. From the results, it can be seen that both external steel rings and ties are effective in improving the stiffness of CFST columns. Particularly, it is evident that the external steel rings are slightly more effective than ties for stiffness enhancement.

4. Theoretical prediction of uni-axial strength of ring-confined CFST columns

4.1 Theoretical model for uni-axial strength prediction

A theoretical model for predicting the axial strength of both unconfined and confined CFST columns was proposed. The model accounts for the confining effect provided by the steel tube and rings on the concrete core as well as that provided by the rings on the steel tube. The enhanced concrete strength is evaluated by combining the models proposed by Han *et al*. (2007), which is to account for the confining effect provided by the steel tube, and Mander *et al*. (1988), which is to account for the confining effect provided by the rings. The external rings will also provide confining effect to the steel tube and hence improve the uni-axial capacity. This effect is represented by the value of a proposed parameter β. The overall formula for predicting the axial strength is shown as follows

$$P_t = f_{cc}A_{cc} + \beta f_y A_s \tag{3}$$

where P_t is the theoretically predicted axial strength of CFST column with external confinement; f_{cc} is the enhanced concrete strength due to both the steel tube and external steel rings; f_y is the yield strength of steel tube; A_{cc} is the area of the concrete core and A_s is the area of the steel tube. The value of β has been obtained by testing bare hollow steel tubes, which were installed with various steel ring spacing of 5*t*, 10*t*, 15*t* and 20*t*, under uni-axial compression. For practical design purpose, the values of β have been correlated to the ratio of confining pressure f_r to concrete cylinder strength f_c'. The confining pressure is evaluated by Eq. (4) with arching effect considered (Mander *et al*. 1988). The resulting equation has been obtained by linear regression analysis and satisfying the boundary condition that β = 1.0 when f_r = 0 as shown in Eq. (5). It should be noted that the value of β should also depend on the steel yield strengths f_y and f_{ys}. However, since the steel yield strength did not vary in the test, the parameter f_y and f_{ys} were not included in the equation. Therefore, the equation is only valid of steel tube of *S*355 grade and mild steel external rings of f_{ys} = 250 MPa.

$$f_r = 0.5 k_e \rho_s f_{ys} \tag{4}$$

$$\beta = 20\varepsilon_c\left(\frac{f_r}{f_c'}\right) + 1 \tag{5}$$

where ε_c (≤0.2) is the axial strain at which the load-carrying capacity is to be predicted, f_r is the confining pressure provided to the in-filled concrete, ρs is the volumetric ratio of the steel rings and f_{ys} is the yield strength of the steel rings.

To estimate the ultimate axial strength of ring-confined CFST columns, the confining effect of the both steel tube and steel rings on the enhanced concrete core strength should be considered. The confining effect provided by the hollow steel tube to concrete core has been studied by Han (2007), and the equations are rewritten as follows

$$\sigma_c = \sigma_o(1-q) + \sigma_o q\left(\frac{\varepsilon_c}{\varepsilon_o}\right)^{0.1\xi} \tag{6a}$$

$$\xi = \frac{A_s f_y}{A_c f_c'} \geq 1.12 \tag{6b}$$

$$q = \frac{0.1\xi^{0.745}}{0.2 + 0.1\xi} \tag{6c}$$

$$\sigma_o = f_c'\left[1.194 + \left(\frac{13}{f_c'}\right)^{0.1\xi}(-0.07485\xi^2 + 0.5789\xi)\right] \tag{6d}$$

$$\varepsilon_o = 1300 + 14.93 f_c' + \left[1400 + \frac{800(f_c - 20)}{20}\right]\xi^{0.2} \tag{6e}$$

where σ_c is the enhanced concrete core strength in the CFST column; ξ is the confinement index of the steel tube; q is a coefficient modifying the confining effect of the steel tube; ε_c is the axial strain of CFST column at peak strength, which is taken as 0.1 for the sake of practical design of axial and flexural members (the strain in real structures rarely exceed 0.1); ε_o is the ultimate compression strain of concrete core and is expressed in terms of micro-strain. The enhanced concrete strength due to both the confining effect provided by the external steel rings and steel tube is then evaluated based on the confined concrete model proposed by Mander *et al*. (1988). The equation is shown as follows

$$f_{cc} = \sigma_c\left(-1.254 + 2.254\sqrt{1 + \frac{7.94 f_r}{\sigma_c}} - 2\frac{f_r}{\sigma_c}\right) \tag{7}$$

where f_r is the confining pressure provided by the steel rings and is given in Eq. (4); k_e is the confinement effectiveness factor; ρ_s is the volumetric ratio of confinement; f_y is the yield strength of steel tube; σ_c is given by Eq. 6(a). The predicted axial strength P_t of confined CFST columns can be obtained by substituting Eqs. (4) to (7) into (3). It can be observed from Eq. (6) that the effect of the external steel rings will reach a maximum at some point beyond which adding more rings would not result beneficial.

4.2 Comparison with experimental results

The validity of the proposed theoretical equations (Eqs. (3) to (7)) is verified by comparing with the experimental results obtained by the authors in this study and by other researchers (Giakoumelis and Lam 2004) on unconfined and confined CFST columns. The comparison is shown in Table 4.

Table 4 Comparison of theoretical and experimental load-carrying capacities

Specimen code	Experimental strength (kN) [1]	Theoretical strength (kN) [2]	[2]/[1]
By authors			
30-5-5*R*	3072	3081	1.00
30-5-10*R*	2592	2570	0.99
30-5-15*R*	2366	2384	1.01
30-5-20*R*	2220	2287	1.03
30-5-00*X*	2015	2087	1.04
Giakoumelis and Lam (2004)			
*C*3	948	960	1.01
*C*5	929	967	1.04
*C*7	1380	1164	0.84
*C*9	1413	1384	0.98
*C*10	1038	1158	1.12
*C*11	1067	1155	1.08
*C*12	998	945	0.95
*C*13	948	942	0.99

For the sake of comparison, the axial strength measured for the CFST columns in both tests at 0.1 axial strain (ε_c) is adopted for comparison. It can be seen from the table that the proposed equations predict very well the axial load-carrying capacity of both unconfined and confined CFST columns tested by the authors and other researchers. The differences are mostly lying within 10%. Therefore, the validity of the proposed equations to predict the axial strength of both unconfined and confined CFST columns is verified.

5. Conclusions

External steel confinement is advocated in this study to improve the imperfect steel-concrete interface bonding of concrete-filled-steel-tube columns during axial compression. Two different forms of external confinement are proposed, which are external steel rings welded to the outer face of the steel tube and steel ties bolted against the external face of the steel tube. The spacing of the external confinement studied was 5*t*, 10*t*, 15*t* and 20*t*, where *t* is the thickness of steel tube equal to 5 mm. All steel confinement adopted were 8 mm diameter with nominal yield strength of 250 MPa.

The effectiveness of the above externally confined CFST columns were studied by carrying out a series of uni-axial compression tests on CFST columns with and without external confinement. A total of 9 CFST columns, which includes 4 columns with external steel rings, 4 columns with external steel ties and a CFST column without external confinement, were tested under uni-axial compression. From the test results, the following observations were obtained:

(1) CFST columns with external confinement (except for ties of 20*t* spacing) were effective in restricting the lateral dilation of the columns and maintain an intact steel-concrete interface bonding. This is verified by the Poisson ratios obtained for these CFST columns, which were about 0.2.

(2) The failure mode of ring-confined CFST columns was the local buckling of steel tube between external steel rings. It is due to the effective and uniform lateral restraint provided by the steel rings against buckling of steel tube at each confinement level.

(3) The failure mode of ties-confined CFST columns was overall buckling of the steel tube, which was firstly initiated at the ties opening near the mid-height of the specimen. This was due to the reduced cross-section moment of inertia and the large splitting tensile stress induced at the tie opening due to compression and subsequent dilation of columns. Consequently, large longitudinal cracks propagated along the height of the specimens as the axial strain increases, which eventually decreases the axial strength dramatically.

(4) The external ring confinement can effectively improve the axial strength of CFST columns by an average value of 23.2%, whereas the tie confinement cannot improve the axial strength.

(5) The external ring and tie confinement can effectively improve the stiffness of CFST columns by an average value of 31.9% and 23.8% respectively.

Lastly, a theoretical model for predicting the axial strength of confined CFST columns has been proposed. The model was developed based on an existing model of unconfined CFST column, but modified to take into account of the confining effect provided by external rings. A new parameter β, which accounts for the enhanced steel strength due to external confinement, was incorporated in the formula for uni-axial strength prediction. An equation for β has been developed based on the test results obtained in a series of uni-axial compression tests on confined hollow steel tube. The model has been compared with the authors' test results and those obtained by other researchers. Good agreement was obtained.

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Notations

A_{cc}	Area of concrete in CFST column
A_s	Area of steel tube
CFST	Concrete-filled steel tube
D	Diameter of CFST column section
D_c	Diameter of external confinement section
E	Measured stiffness of CFST column without external confinement
E_c	Measured stiffness of CFST column without external confinement
f_c'	Uni-axial concrete compressive strength represented by cylinder strength
f_r	Confining pressure provided by external confinement
f_y	Yield strength of steel tube
f_{ys}	Yield strength of steel rings
HSC	High-strength concrete
H	Length of CFST column
LVDT	Linear variable displacement transducer
NSC	Normal-strength concrete
P	Measured axial strength of CFST column without external confinement
P_c	Measured axial strength of CFST column with external confinement
RC	Reinforced concrete
s	Spacing of external confinement
t	Thickness of steel tube
ρ_s	Volumetric ratio of external confinement
ω_E	Stiffness enhancement ratio
ω_S	Axial strength enhancement ratio