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Axial behavior of RC columns strengthened with SCC filled square steel tubes

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Abstract. Self-compacting Concrete (SCC) Filled Square steel Tubes (SCFST) was used to strengthen square RC columns. To establish the efficiency of this strengthening method, 17 columns were tested under axial compression loading including 3 RC columns without any strengthening (WRC), 1 RC column strengthened with concrete jacket (CRC), 13 RC columns strengthened with self-compacting concrete filled square steel tubes (SRC). The experimental results showed that the use of SCFST is interesting since the ductility and the bearing capacity of the RC columns are greatly improved. The improvement ratio is significantly affected by the nominal wall thickness of steel tubes (t), the strength grade of strengthening concrete (C), and the length-to-width ratio (L / B) of the specimens. In order to quantitatively analyze the effect of these test parameters on axial loading behavior of the SRC columns, three performance indices, enhancement ratio (ER), ductility index (DI), and confinement ratio (CR), were used. The strength of the SRC columns obtained from the experiments was then employed to verify the proposed mode referring to the relevant codes. It was found that codes DBJ13-51 could relatively predict the strength of the SRC columns accurately, and codes AIJ and BS5400 were relatively conservative.

Keywords: RC columns; self-compacting concrete (SCC); square steel tubes; strengthening method; axial load

1. Introduction

Making existing reinforced concrete buildings damaged by corrosion or accidents conform to new safety regulations and functions are a tremendous technological and financial challenge, which poses a huge socio-economic problem in the next few decades (Colomb *et al*. 2008).

Currently, common techniques for strengthening RC structures include concrete jacketing (Colomb *et al*. 2008) and FRP confinement (Teng *et al*. 2003, Chen and Teng 2003, Dai *et al*. 2011, Bai *et al*. 2014), and steel jacketing (Xiao and Wu 2003). Meanwhile new strengthening techniques and composite techniques are carried out continuously, but using steel jacketing to enhance the strength of RC column and to improve the deformability is drawing increasing

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interest.

Many researchers have been conducted to investigate the behavior of using a steel jacket to strengthen RC columns since 1960s. Aboutaha (Aboutaha and Machado 1999, Aboutaha *et al.* 1999a, b) tested a system which combined a relatively thin rectangular jacket with a through bolt and showed enhanced confinement efficiency. And this method was effective to improve the ductility of columns with inadequate shear resistance when the jacket was only designed for potential plastic hinge regions. Xiao and Wu (2003) and Abedi *et al.* (2010) investigated several RC columns strengthened with square steel jacket and additional stiffeners welded to the potential plastic hinge regions. The results validated the efficiency of the partially stiffened steel jacket, which not only prevented brittle shear failure but also greatly improved the hysteretic behavior and the ductility of the columns with achieving an ultimate drift ratio of more than 8%. However, for these steel jacket strengthening methods, the researchers usually connected the steel jackets and the RC column by bolts, adhesive, cement grout, etc. As a result, the efficiency of enhanced confinement offered by the steel jackets to RC column is reduced because of the poor bond strength. Furthermore, the RC columns requiring enhanced flexural stiffness will be not provided unless using thick-walled steel. In addition, they are not applicable for strengthening circular columns.

Therefore concrete filled steel tubes (CFT) strengthening method has been proposed to improve the strengthening performance. The procedure is to strip off the protective layer and deficient section of RC column firstly, and then pack the steel tube welded by two pieces of L-shaped or semicircular steel plates, and pour into concrete lastly to make them work together. An effective confinement will be offered by the steel tube which causes the core concrete to behave in a triaxial stress state while the filled concrete prevents the wall of the steel hollow section from buckling inward. Consequently, both strength and ductility of the concrete are enhanced and the flexural stiffness of the RC column will be significantly increased with small cross-section changes. Moreover the CFT strengthening method can be widely applied to strengthening square, circular or novel form RC columns. Priestley *et al.* (1994a, b) proposed the method of square RC columns strengthened with concrete filled elliptical steel tubes (CFET). The experimental results indicated that the lateral stiffness of the strengthening columns was increased by an average of 64% and the ductility was significantly improved. Nevertheless, the elliptical tube would cause the section of the columns changing substantially. Thus it may not be favorable for retrofitting rectangular or square columns perfectly from the architectural and functional points of view. Miller (2006), Sezen and Miller (2011), Wang (2011) and Zhou *et al.* (2012) conducted tests on circular RC columns strengthened with concrete filled circular steel tube (CFCT). Compared with the experimental results of concrete jacket and FRP jacket strengthening tests, CFCT strengthening is more effective to improve the specimen stiffness, member strength and ductility because of the existence of sufficient confinement. Furthermore, CFCT strengthening method requires less construction time and costs because the tube can serve as formwork.

However, most of the research work focused on CFCT strengthening methods while little research focused on concrete-filled square steel tubes (CFST). It is mainly because the confinement provided by square steel tube is less effective than that of circular steel tubes. Nevertheless, the use of CFST is gradually being accepted because of its convenient construction and simple node structure (Han 2011). Moreover from the architectural point of view, square steel tubes are more appropriate to strengthen square RC columns which widely exist in engineering practice as columns. In addition, it has been found from the research of Miller (2006) that the gap between the formwork (steel tubes) and RC columns is so narrow to vibrate concrete

(a) (b)

Fig. 1 Improper compaction of the cover concrete in Miller (2006)

that the formwork cannot be filled uniformly by Normal Vibrated concrete (NVC). Consequently, the columns displayed many surface voids from the improper compaction cover after the formwork was removed as shown in Fig. 1. It seriously reduced the bond strength to make the confinement ineffective. So the Self-Compacting Concrete (SCC) is used which allows pouring concrete easily without vibration even in the presence of a highly dense reinforcement or novel form of construction (Muciaccia *et al*. 2011, Holschemacher 2004). Nevertheless, SCC displays higher shrinkage than NVC due to the higher volume of paste (Loser and Leemann 2009). So the right amount of concrete expansion agent is added to compensate for the shrinkage of SCC and enhanced the bond strength (Chang *et al*. 2009).

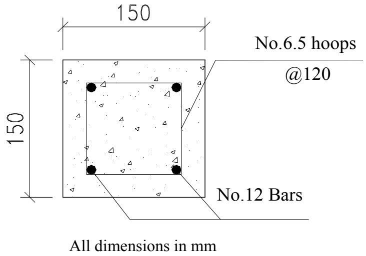
Therefore, this paper performs an experimental investigation on the axial behavior of square RC columns strengthened with SCC-filled Square Steel Tubes. The failure mode, the axial deformation behavior, the enhanced strength ratio and the ductility of the SCFST columns are investigated to establish the advisability of this strengthening method. Moreover, experimental results are employed to verify the proposed equations referring to the CFSTs design codes AIJ, BS5400, and DBJ13-51-2003.

2. Experimental program

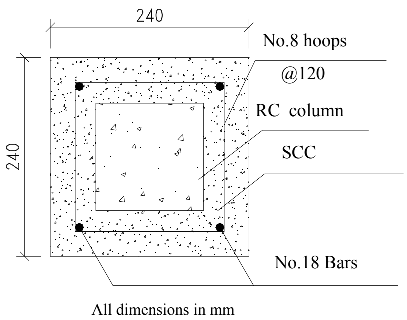
2.1 Test specimens

In this experimental program, 17 specimens were tested under axial compression loading including 3 RC columns without strengthening (WRC), 1 RC column strengthened with concrete jacket (CRC), 13 RC columns strengthened with self-compacting concrete-filled square steel tubes (SRC). The main parameters for the tests are as follows: (1) the nominal wall thickness of steel tubes (t), which varies from 2 mm to 4 mm; (2) the designed strength grade of strengthening concrete (C), which includes C40, C50 and C60; (3) the length-to-width ratio (L / B) of the specimens, which changes from 3 to 9. Following the code AIJ (2008) and DBJ13-51-2003, the CFST with $L / B \leq 4$ is regarded as stub column, and $L / B > 4$ is slender column where second-order effect could not be neglected.

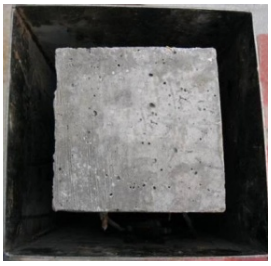
The height of the stub columns is 0.72 m, and that of the slender SRC columns varies from 1.2 m, 1.5 m, 1.8 m, to 2.1 m. The cross section details of all the specimens are shown in Fig. 2. The cross section of the SRC is designed smaller than that of the CRC column. All the test parameters



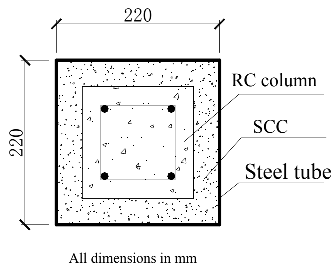
(a) Cross section of WRC



(b) Cross section of CRC



(c) Before pouring concrete



(d) Cross section of SRC

Fig. 2 Cross section details of all columns

Table 1 Geometrical and material parameters for all test specimens

Specimen	B × *t* × L (mm)	Concrete grade	Steel consumption (mm^3)	*L* / *B*	N_e (kN)
WRC0-C25-0.72	150 × 0 × 720	C25	-	5	685
WRC0-C25-0.72	150 × 0 × 720	C25	-	5	670
WRC0-C25-1.8	150 × 0 × 1800	C25	-	12	660
CRC0-C50-0.72	240 × 0 × 720	C50	1.15×10^6	3	2020
SRC2-C50-0.72	220 × 1.78 × 720	C50	1.12×10^6	3	2217
SRC3-C50-0.72	220 × 2.80 × 720	C50	1.75×10^6	3	2445
SRC4-C50-0.72	220 × 3.80 × 720	C50	2.37×10^6	3	2650
SRC3-C40-0.72	220 × 2.80 × 720	C40	1.75×10^6	3	2320
SRC3-C60-0.72	220 × 2.80 × 720	C60	1.75×10^6	3	2620
SRC3-C50-1.2	220 × 2.80 × 1200	C50	1.92×10^6	6	2290
SRC3-C50-1.5	220 × 2.80 × 1500	C50	3.65×10^6	7	2230
SRC3-C50-1.8	220 × 2.80 × 1800	C50	4.38×10^6	8	2170
SRC3-C50-2.1	220 × 2.80 × 2100	C50	5.10×10^6	9	2050
SRC2-C50-1.8	220×1.78×1800	C50	2.80×10^6	8	2030
SRC4-C50-1.8	220 × 3.80 × 1800	C50	5.93×10^6	8	2400
SRC3-C40-1.8	220 × 2.80 × 1800	C40	4.38×10^6	8	2070
SRC3-C60-1.8	220 × 2.80 × 1800	C60	4.38×10^6	8	2275

Table 2 Material properties of steel

steel	t or D (mm)	f_y (MPa)	Es (GPa)	Elongation rate (%)
Steel tube	1.78	307.1	202	22
Steel tube	2.80	280.3	205	25
Steel tube	3.80	265.2	209	28
Hoop Reinforcements	6.5	310.4	189	25
Hoop Reinforcements	8	286.6	208	20
Longitudinal Reinforcements	12	384.2	195	22
Longitudinal Reinforcements	18	437.1	197	23

Table 3 Mix proportions (kg/m^3) and compressive strength of concrete (MPa)

Concrete grade	Water	42.5R cement	River sand	Coarse aggregate	Expansive agent	Water reducer	Fly ash	f_{cu}	f_c	f_c'
C25 NVC	218.8	336.7	733.9	1198.5	-	-	-	32.6	21.8	26.5
C40 SCC	184.7	432.5	784.5	953.2	1.7	48.0	83.5	48.8	32.7	38.8
C50 SCC	193.9	484.7	737.7	927.3	1.9	49.4	95.0	52.1	34.9	42.1
C60 SCC	179.0	487.8	715.6	969.7	2.0	48.0	92.7	61.1	40.9	50.8

for each specimen are summarized in Table 1. The nomenclature followed in the tests is: XRCt-C-L (i.e., SRC3-C50-0.72), where X stands for strengthening method, "t" is the nominal wall thickness of steel tube in mm, "C" is the designed strength grade of strengthening concrete in MPa, and "L" means the nominal length in meters.

2.2 Material properties

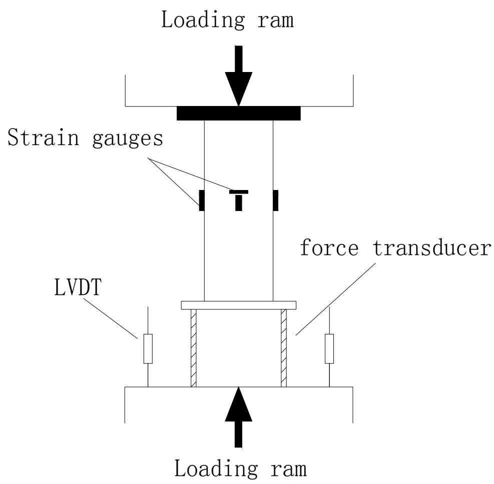
2.2.1 Steel

The RC columns were reinforced with four 12 diameter longitudinal bars, and were transversely reinforced with 6.5 mm diameter hoops, spaced at 120 mm. The strengthening section of CRC columns were reinforced with four 18 diameter longitudinal bars, and were transversely reinforced with 8 mm diameter hoops, spaced at 120 mm. The total steel consumption was 1.15 × 106 mm^3 which was slightly larger than that of SRC2-C50-0.72 as shown in Table 1. For SRC columns, two pieces of L-shaped steel plates which were accurately cut and machined to the required length and thickness were butt welded into the square steel jacket. The coupons were taken from the L-shaped steel plates and reinforcement, and tensile tests on these coupons were conducted to measure material properties. The average yield strength (f_y) of the steel, the modulus of elasticity (Es) and elongation rate obtained from the tests are shown in Table 2.

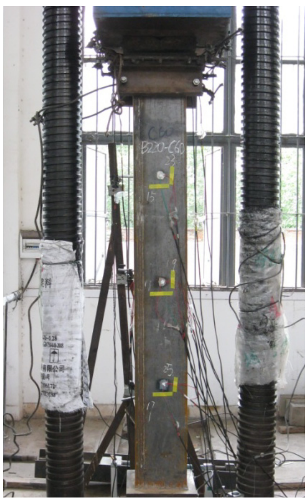
2.2.2 Concrete

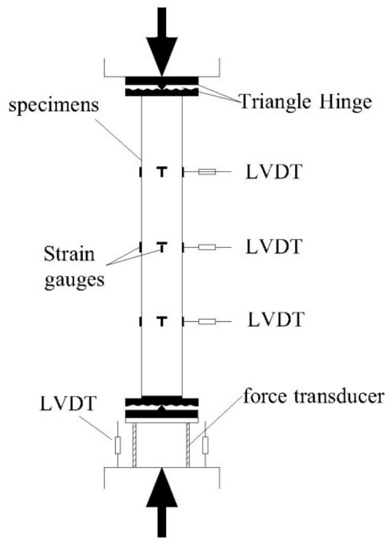
The mix proportions used in this paper, which were determined by trial mixtures, are summarized in Table 3. Grade 42.5R ordinary Portland cement, Class 2 fly ash, river sands with the maximum size of 5 mm, coarse aggregate with a maximum size of 15 mm, U-type expansive agent and polycarboxylic acid water reducer were used. Slump flow of the SCC was 270 mm. As model columns simulating deficient columns, the RC columns were poured with NVC whose nominal compressive strength was low to 25 MPa. The strengthening concrete of the CRC was cast with SCC of which the nominal compressive strength was 50 MPa, while that of the SRC columns varied between 40, 50 and 60 MPa. The cube concrete compressive strength 'f_{cu}' is determined by testing the cube specimens of dimensions150 × 150 × 150 mm after 28 days of curing. 'f_c' is the prism compressive strength ($f_c = 0.67f_{cu}$) and 'f_c' is the cylinder compressive strength equivalent translated by Eurocode 2. All of these are listed in Table 3.

2.3 Test setup and instrumentation layout



(a) Setup and instrumentation of stub column test





(b) Setup and instrumentation of slender column test

Fig. 3 Test setup and instrumentation

All the tests were performed on a 5000 kN capacity universal testing machine. The test setup and instrument layout are shown in Fig. 3. For the stub columns tests, the specimens were placed into the testing machine and two thick stiff plates were placed on the ends of the specimens to ensure the axial load applied simultaneously to the square steel tubes and the concrete core as shown in Fig. 3(a). For the slender columns tests, the axial load was applied through a desired pinned support simulated by a triangular hinge which allowed the specimen to rotate but restrained its translation at the same time.

A force transducer was placed below the bottom to accurately measure the applied axial load in real-time. Two linear variable displacement transducers (LVDTs) were used to measure the axial deformation. For the slender columns tests, three additional LVDTs symmetrically measured the lateral deflection of the slender columns at the mid length (0.5 L) and quarter-heights (0.25 L, 0.75 L). Eight electrical strain gauges were glued to the external surface of the square steel tubes at mid-height to measure the axial and transverse strains in four locations 0°, 90°, 180°, and 270°. A computerized data-acquisition system was used to collect the experimental data of the load, deformation and strain.

3. Experimental results and analysis

3.1 Failure mode

The typical failure modes include the material failure, the outward local buckling of steel tubes, and the overall flexural buckling in this experiment as shown in Figs. 4 and 5, which vary depending on type of strengthening methods and value of L/B.

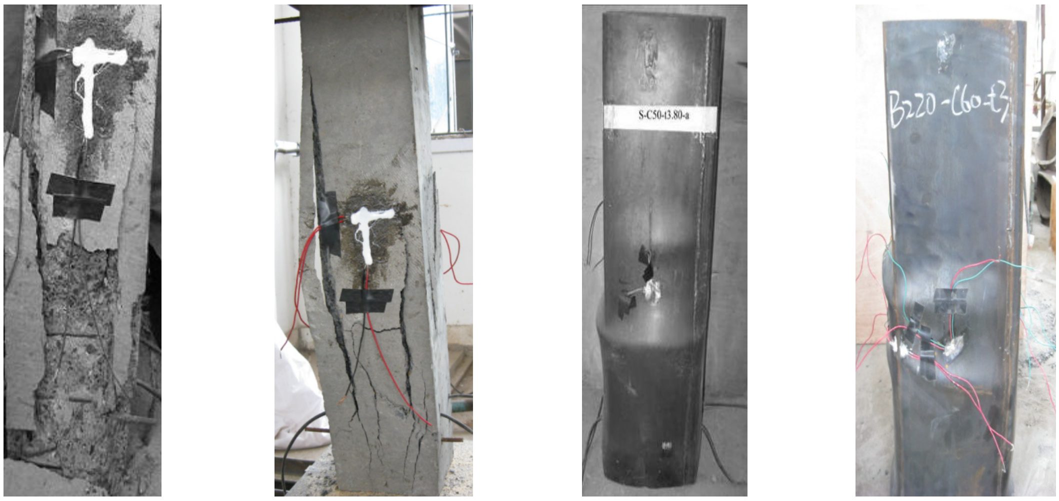
3.1.1 Stub column tests

For the WRC columns, the typical failure mode was the material failure as shown in Fig. 4(a). First phenomenon was observed as longitudinal cracking of the cover concrete as the applied load increased up to 70% of the maximum load. As the load increased, the cover concrete was spalling and flaking due to the development of cracks in the concrete surrounding the longitudinal reinforcements. And then, the longitudinal reinforcements were serious buckling coincided with the failure of the specimen. For the CRC columns, the failure mode was similar to the WRC columns as shown in Fig. 4(b). The maximum longitudinal deformation of WRC and CRC was 1.6 mm and 3.0 mm respectively.

For SRC columns, the tested stub columns behaved in a relatively ductile manner. The typical failure mode was the outward local buckling of steel tubes because of the stability supplied by the infill of SCC concrete as clearly shown in Figs. 4(c) and (d). The local buckling was firstly observed about 70% of the maximum load and became more serious as the load increased. After the ultimate load, the applied load maintains at a certain loading level while the deformation is still aggravating. When the test was terminated, two obvious bulges were observed along the height as shown in Fig. 4(c) and (d). The longitudinal deformation of the SRC columns was up to even beyond 6 mm which shown that SCFST could increase the deformation capacity of the RC columns. The failure mode is similar to that of the CFSTs as observed by Han and Yao (2004).

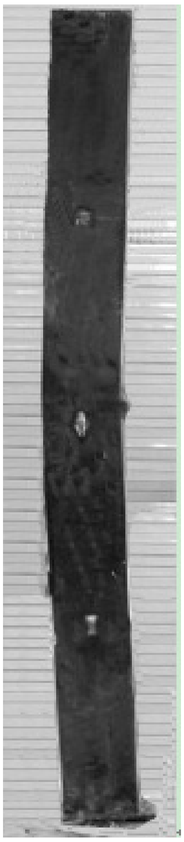
3.1.2 Slender column tests

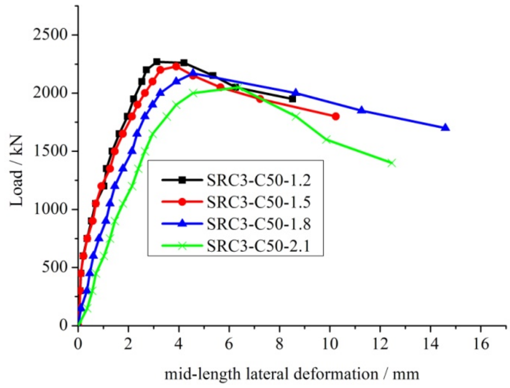
For the tested slender columns, the typical failure mode was the overall flexural buckling which



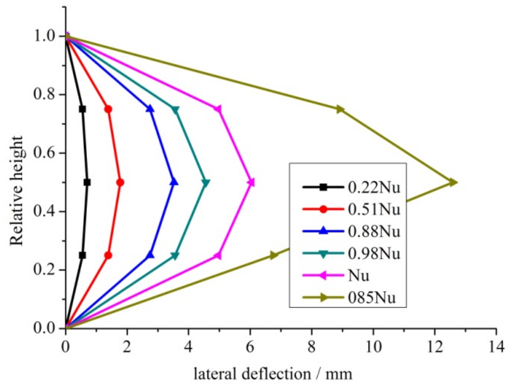
(a) WRC0-C25-0.72 (b) CRC0-C50-0.72 (c) SRC4-C50-0.72 (d) SRC3-C60-0.72

Fig. 4 Typical failure modes of stub columns





(b) Load versus mid-length lateral deflection curve



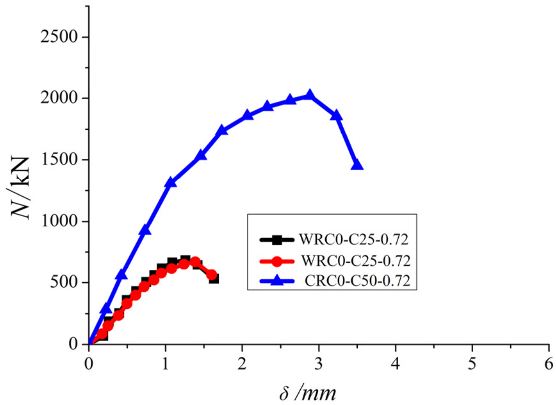
(a) Typical failure modes of slender columns (c) Load versus overall-lateral deflection curve

Fig. 5 Deformation process of slender columns occurred close to the centre of the columns as shown in Fig. 5(a). From the load (*N*) versus the mid-length lateral deflection (*f*) curve of the slender columns as shown in Fig. 5(b), the columns were unable to maintain a perfect straight and the lateral deflection would occur due to the presence of the initial imperfections even when the applied load was small. The lateral deflection would generate a secondary moment which could conversely further increase the lateral deflection. Eventually led to an instability problem and made axial load capacity of the composite columns decrease seriously. And the displacements significantly increased as the length of the column and the applied load further increasing as shown in Figs. 5(b) and (c) respectively. The effect of second-order moment would be prime importance and cause the slender column to fail by bending just like a half-wave sine curve rather than by compression. The failure mode is similar to that of the CFSTs as observed by Han (2000) and Yu *et al*. (2008).

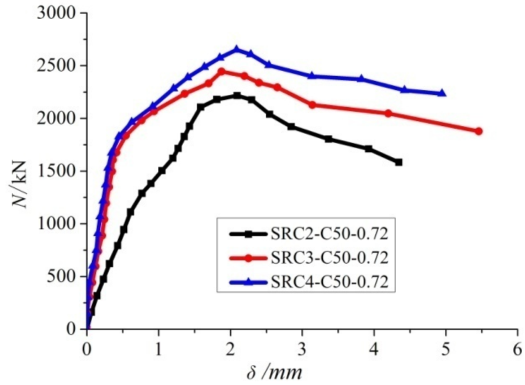
3.2 The load (N) versus axial deformation (δ) curves

The load (N) versus axial deformation (δ) curves of all the tested columns are shown in Fig. 6, where δ is an average value measured from LVDTs. In the initial stage of loading, the N-δ curve is close to linearity before the applied load is up to approximately 70% of the ultimate load for the SRC columns generally. In this stage, all the members are in elastic stage and the Poisson's ratio of steel (about 0.25~0.30) is greater than that of concrete (about 0.17~0.20). Thus, the lateral expansion of concrete is smaller than the steel tube under the same longitudinal deformation so that the steel tubes have no confining effect on the concrete core at the initial stage of loading. Beyond this load level, the steel tubes gradually go into elastic-plastic state, and its modulus decreases significantly. The N-δ curve diverges from its initial linearity. However the modulus of concrete decreases slightly and the stress is redistributed between the steel tubes and the concrete persistently. The stress on the concrete increases significantly so that Poisson’s ratio of concrete increases even beyond 0.5. At this stage, the lateral expansion of concrete core gradually catches up with that of steel. Therefore, a radial stress develops at the steel-concrete interface, which causes the concrete core to be subjected to triaxial stress, thereby enhancing the concrete strength and confining the deformation. It is also the main reason of the load being kept constant at a certain loading level after the ultimate load for SRC columns. Comparing Fig. 6(a) with Fig. 6(b) and Fig. 6(c), the SRC columns have a superior performance to CRC columns on the axial load-bearing capacity and ductility.

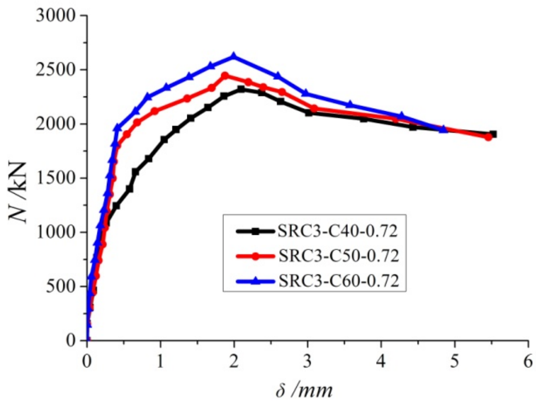
Figs. 6(b) and (e) show the comparison of N-δ curves for the stub and slender specimens in different wall thickness of the steel tubes (t). It is clear that the axial load-bearing capacity and the deformation capacity increase as the wall thickness increases, which indicates the more effective constraints can be obtained by the thicker steel tubes. Figs. 6(c) and (f) show the comparison of N-δ curves for the stub and slender specimens in different strength grade (C). It can be seen that the axial load-bearing capacity increases but the deformation capacity decreases as C increases. This is mainly determined by the properties of concrete as the high strength concrete has an extremely brittle. Meanwhile, it is also found that the axial load-bearing capacity of the SRC columns with length l = 0.72 m is far higher than that of length l = 1.8 m. The reason can be seen from the above failure mode. The slender columns failed by bending moment rather than by material compression failure because of the instability problem caused by second-order moment. The member strength cannot be made full use of and the confinement is ineffective. Moreover, the axial load-bearing capacities of the slender columns decrease with the increase of length as shown in Fig. 6(d) because the second-order moment increases significantly as the length of the column increasing.



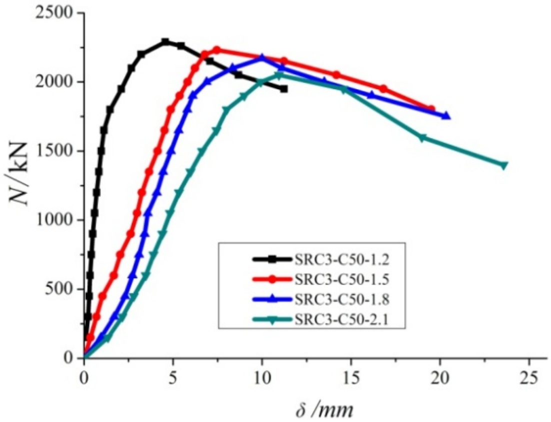
(a) strengthening method



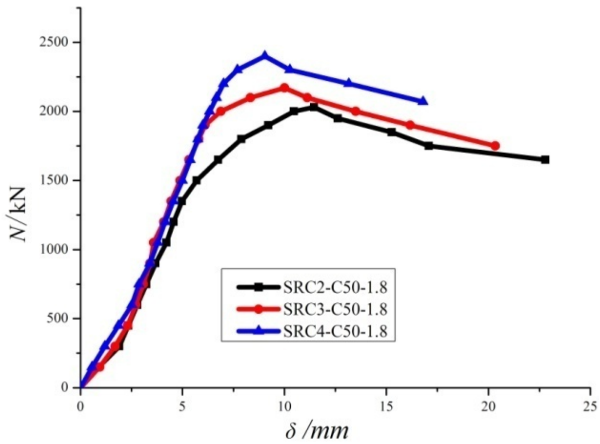
(b) wall thickness of the steel tubes



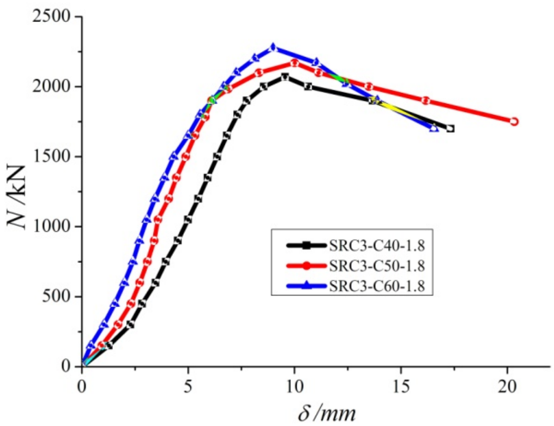
(c) strength of strengthening concrete



(d) height of SRC columns



(e) wall thickness of the steel tubes



(f) strength of strengthening concrete

(1) Stub columns

(2) Slender columns

Fig. 6 Comparison of N-δ curve of the test specimens

4. Performance indices

To quantitatively analyze the effectiveness of SCFST, three numerical performance indices, enhancement ratio (ER), ductility index (DI), and confinement ratio (CR), are used to represent the degree of improving bearing capacity of the deficient columns, the ability of plastic deformation after the ultimate load, and the efficiency of enhanced confinement offered by the steel tubes, respectively.

4.1 Enhancement ratio (ER)

The degree of improving bearing capacity of the deficient columns is one of the most interesting indices to evaluate the efficiency of a strengthening method. In this paper ER is defined as the ratio between the maximum load of the SRC columns, or the CRC column and the WRC columns

$$ER = \frac{N_{\max,XRC}}{N_{\max,WRC}} \tag{1}$$

Fig. 7 presents the ER in terms of strengthening methods, C and t for the specimens with a length l = 0.72 m or l = 1.8 m. As can be seen that the ER of SRC2-C50-0.72 is 9.7% more than that of CRC with less steel consumption (SRC2: 1.12 × 106 mm^3, CRC: 1.15 × 106 mm^3) and smaller cross-section (SRC2: 25900 mm^2, CRC: 35100 mm^2). It shows that the SCFST strengthening method can improve the load-bearing capacity of RC columns more effectively.

It also can be noted that the ER value increases with the increase in t and C for the SRC columns with length l = 0.72 m or l = 1.8 m. For the stub columns (l = 0.72 m), the ER of SRC3-C50, SRC4-C50 are 10.3% and 19.5% respectively more than that of SRC2-C50. And the ER of SRC3-C50, SRC3-C60 are 5.4% and 13% respectively more than that of SRC3-C40. For the slender columns (l = 1.8 m), SRC3-C50, SRC4-C50 have obtained a higher ER value of 6.9% and

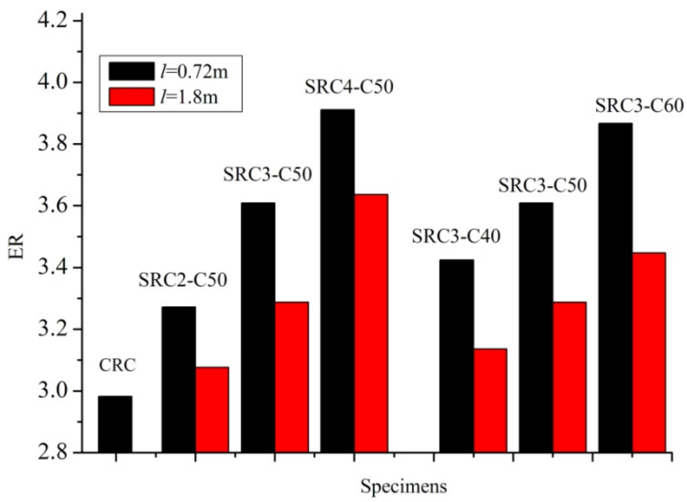


Fig. 7 ER for specimens with L = 0.72 m or L = 1.8 m

18.2% respectively compared with SRC2-C50. And SRC3-C50, SRC3-C60 have obtained a higher ER of 4.8% and 9.9% respectively compared with SRC3-C40. It means that the higher load carrying capacity of the SRC columns can be gained by thicker steel tubes and higher strength concrete, and the increasing degree is relatively large within the scope of the current research. Furthermore, the wall thickness of the steel tubes has a more significant effect than concrete strength.

It is also apparent that the ER of SRC columns with $l = 1.8$ m is at least 6.4% and at most 12.2% less than that of SRC columns for $l = 0.72$ m in case of the same t and C, which means L/B has a remarkably detrimental effect on the load-bearing capacity of the SRC columns.

4.2 Ductility index (DI)

The ductility of the SRC columns is one of the most interesting advantages in the comparison of the CRC column. To quantify the enhancement of SCFST on the ductility of the stub column specimens, ductility index (DI) is defined as the ratio between the displacement corresponding to 85% of the maximum load (in the descending branch) and the displacement from the maximum load

$$DI = \frac{\delta(0.85N_{\max})}{\delta(N_{\max})} \tag{2}$$

Fig. 8 presents the DI in terms of strengthening methods, C and t for columns of length $l = 0.72$ m. As being shown that the ductile behavior of the WRC columns (DI = 1.182) is significantly improved by SCFST strengthening method (DI $\geq$ 1.691) while concrete jacket does not (DI = 1.153), and a higher DI can be obtained with the lower strength of concrete and thicker steel tubes. The DI of SRC3-C50, SRC4-C50 are 26.5% and 30.7% respectively more than that of SRC2-C50. And the DI of SRC3-C50, SRC3-C60 are 2.8% and 25.4% respectively less than that of SRC3-C40. It means a thinner wall of the steel tubes has a poorer ductility. And the high strength

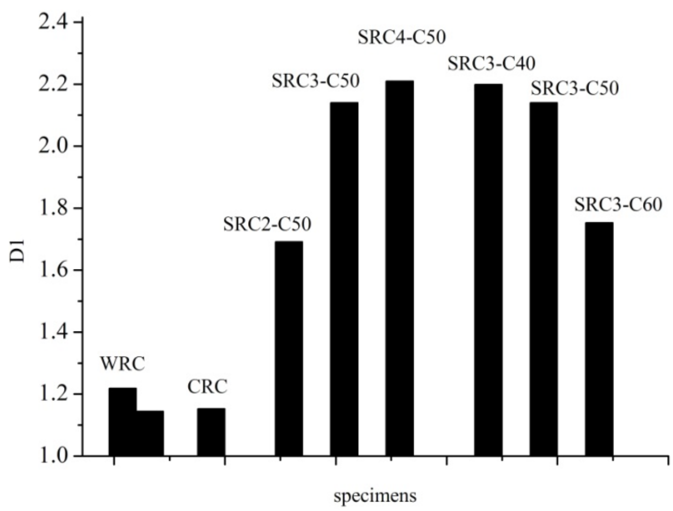


Fig. 8 Ductility index (DI) for specimens with $l = 0.72$ m

concrete usually defined by cube compressive strength (f_{cu}) above 60 MPa has an extremely poor ductility. Thus many design codes such as Eurocode 4 restrict the maximum of B/t and limit the use of high strength concrete to guarantee a good ductile behavior of the CFT columns, therefore their safety.

4.3 Confinement ratio (CR)

Comparing with concrete jacket, the RC columns strengthened with SCFST have higher load-bearing capacity (as shown in Fig. 7) and higher ductility (as shown in Fig. 8) mainly due to the confinement offered by the steel tubes. If we ignore the effect of confinement, SRC columns can be regarded as normal reinforced concrete structures, and the nominal axial capacity (N_u) can be calculated using Eq. (3). If not, the real value is expressed by an experimental value (N_e) .So CR calculated using Eq. (5) is used to represent the confinement.

$$N_u = \varphi \cdot (A_c f_c + A_s f_s) \tag{3}$$

$$\varphi = \begin{cases} 1 & (L/B \le 8) \\ 0.98 & (8 < L/B \le 10) \end{cases} \tag{4}$$

$$CR = \frac{N_{\max,SRC}}{N_{\max,WRC} + N_u} \tag{5}$$

Where φ is the stability factor of axially loaded reinforced concrete columns which can be calculated by Eq. (4); Ac and As are cross-sectional area of strengthening concrete and steel tube respectively in the axial direction; f_c and f_s are prism compressive strength of strengthening concrete and yield strength of steel tubes respectively.

Fig. 9 presents the CR for the SRC columns with a length l = 0.72 m or l = 1.80 m. It denotes that the steel tubes can offer an efficiently enhanced confinement to the concrete core for SRC stub columns, while the confinement is so small that it should be ignored for the SRC slender columns. The average of CR for the test stub specimens is 1.12, which means this composite structure has at least an extra 12% amplification except for taking full advantage of the strength of material and RC columns. In contrast, due to the effect of second-order moment, the average of CR is 1.02 and the minimum (SRC3-C40-1.8: 0.995) is less than 1 for the test slender specimens, which means the enhanced confinement is very small. The similar conclusions have been made by Dundu (2012) for CFSTs. Moreover, the confining effect between steel and concrete is neglected in EC4 proposals when the relative slenderness ratios exceed 0.5 (approximately corresponds to L/B = 12).

5. Prediction of the strength

Currently, there are few researches on the predicted load-bearing capacity for RC columns strengthened with CFT, while most theoretical and experimental researches have been carried out on CFTs. Even different specific codes for the design of CFTs have been formulated and practiced in the respective countries over several decades. The mechanical properties and failure modes of the SRC columns are similar to that of the CFTs. Thus Miller (2006) attempted to use the strength equations of CFCTs to predict the load capacity of RC columns strengthened with CFCT and found Cai's equations could relatively accurately predict the results which were within 10% of the experimental columns strength. Therefore to obtain a relatively accurate design method, four simple calculation models referring to the CFST codes, AIJ, BS5400, EC4, and DBJ13-51, are used to predict the strength of the SRC columns. Each of these codes reflects the design philosophies in the respective countries so that there are many differences in the predictions.

The predicted strength (N_p) is divided into two parts: the strength of RC columns affected by the confinement of steel tubes (N_1) and that of the retrofitted section (N_2). However, the confinement only has an effect on the concrete but on the longitudinal reinforcements for RC columns. So the N_1 is calculated by sum of the strength of longitudinal reinforcements and the effected strength of concrete which refers to the codes. The strengthening section is simplified to a CFST column and its calculation formula also refers to the codes. Comparisons of the predicted strength (N_p) based on different code provisions with the experimental results (N_e) for SRC stub columns are shown in table 4, and the average and variation coefficient of the ratio N_p / N_e are also presented. For the test slender columns, the strength calculation could refer to codes of normal reinforced concrete slender columns and will not be discussed in this paper because of the so small confinement from the study mentioned above.

It is found that the formula based on code DBJ13-51 is more accurate than the others with an average N_p / N_e ratio of 1.013 and a variation coefficient of 0.014 mainly because the confining effect offered by the square steel tubes is considered. Certainly to achieve the aims of serviceability and safety, a reduction factor need be applied to the material properties when it is used in engineering practice. The other codes neglect the confining effect between steel and concrete, thereby superimpose the strength of both the concrete and steel sections. And accounting for the effects of creep, the use of uncracked concrete section and etc, loads and materials properties are significantly reduced in different degrees by applying a partial safety factor in codes AIJ and BS5400. Consequently, the calculation models referring to the codes are on the safe side for predicting the strengths which gives conservative with an average of 8 and 28% lower than the test results, respectively.

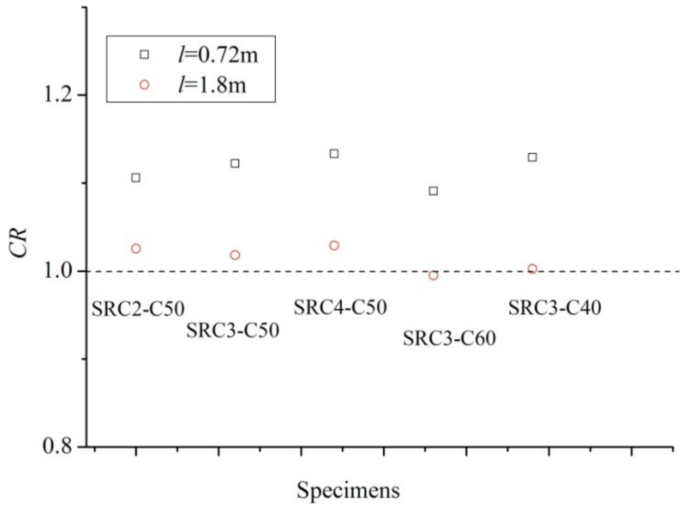


Fig. 9 Confinement ratio (CR) for specimens L = 0.72 m or L = 1.8 m

Table 4 Comparison of test and calculated results

Codes	Calculation formula	Specimens	N_p	N_e	N_p / N_e	Average	Standard deviation	Variation coefficient
DBJ 13-51	$N_p = N_1 + N_2$ $N_1 = (1.18 + 0.85\xi)\, A_{c1}f_{c1} + A_{s1}f_{y1}$ $N_2 = (1.18 + 0.85\xi)\,(A_{c2} + A_{s2})\, f_{c2}$	SRC2-C50-0.72	2243	2217	1.012	1.013	0.014	0.014
		SRC3-C50-0.72	2473	2445	1.011			
		SRC4-C50-0.72	2627	2650	0.991			
		SRC3-C40-0.72	2405	2320	1.037			
		SRC3-C60-0.72	2659	2620	1.015			
AIJ	$N_p = N_1 + N_2$ $N_1 = A_{s1} \cdot f_{y1} + 0.85 f'_{c1} \cdot A_{c1}$ $N_2 = A_{s2} \cdot f_{y2} + 0.85 f'_{c2} \cdot A_{c2}$	SRC2-C50-0.72	2029	2217	0.915	0.916	0.015	0.016
		SRC3-C50-0.72	2241	2445	0.917			
		SRC4-C50-0.72	2361	2650	0.891			
		SRC3-C40-0.72	2175	2320	0.938			
		SRC3-C60-0.72	2415	2620	0.922			
BS 5400	$N_p = N_1 + N_2$ $N_1 = A_{s1} \cdot f_{y1} + 0.45 f_{cu} \cdot A_{c1}$ $N_2 = A_{s2} \cdot f_{y2} + 0.45 f_{cu} \cdot A_{c2}$	SRC2-C50-0.72	1528	2217	0.689	0.723	0.023	0.032
		SRC3-C50-0.72	1764	2445	0.721			
		SRC4-C50-0.72	1993	2650	0.752			
		SRC3-C40-0.72	1729	2320	0.745			
		SRC3-C60-0.72	1859	2620	0.709			

* A_{c1}, A_{s1} = concrete and steel cross-sectional area of RC column. A_{c2}, A_{s2} = concrete and steel tubes cross-sectional area of strengthening section. f_{c1}, f_{c2} = prism compressive strength of RC column concrete and strengthening concrete. f_{y1}, f_{y2} = yield strength of longitudinal reinforcements and strengthening steel tubes. The confinement index $\xi = A_{s2}f_{y2} / (f_{c1} \cdot A_{c1} + f_{c2} \cdot A_{c2})$.

6. Conclusions

The experimental results of 17 RC columns strengthened with different strengthening method under axial load are presented. It has been shown that the SCFST strengthening method is more effective since the bearing capacity of the SRC columns is 9.7% greater than that of the CRC columns with less steel consumption and cross-section. Moreover the ductile behavior of the RC columns is improved by SCFST approximate to 50% while concrete jacket strengthening method cannot improve it.

The performance of RC columns strengthened with SCFST is significantly affected by three parameters: wall thickness of the steel tube (t), designed strength grade of strengthening concrete (C) and length-to-width ratio (L / B) of the specimens. The following conclusions are reached within the scope of the current research.

- The wall thickness of steel tubes has a significant effect on both bearing capacity and ductility of the SRC columns. And the increasing degree is relatively large within the scope of the current research.
- The bearing capacity of the strengthening columns increases with C increasing, but the degree of the increase is less obvious than the wall thickness of the steel tubes. And the ductility significantly decreases with the strength increasing, especially in the use of high strength concrete. The ductility of the SRC columns with C60 strength grade concrete is

25.4% less than that of C40.

- L/B has a remarkably detrimental effect on the behavior of the SRC columns. When L/B is large (the value is approximately 8 in the experiments), an average value of 1.02 for the enhanced confinement is so small that it should be ignored for the slender columns. In order to take full advantage of the confinement effect between steel tubes and concrete, SRC columns with large L/B are not recommended to be used in practices.

The simple calculation models for CFST columns referring to the relevant codes were used to predict the strength of the SRC columns. In contrast with the previous results obtained from the experiments, the prediction of code DBJ13-51 is more accurate while that of code AIJ and BS5400 is relatively conservative.

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