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Tests and calculations for hollow structural steel (HSS) stub columns filled with self-consolidating concrete (SCC)

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Abstract

The behaviour of self-consolidating concrete (SCC) filled hollow structural steel (HSS) stub columns subjected to an axial load was investigated experimentally. A total of 50 specimens were tested. The main parameters varied in the tests are: (1) sectional types: circular and square; (2) steel yielding strength: from 282 to 404 MPa; and (3) tube diameter or width to wall thickness ratio (D/t or B/t): from 30 to 134.

A mechanics model is developed in this paper for concrete-filled HSS stub columns. A unified theory is described whereby a confinement factor (ξ) is introduced to describe the composite action of the steel tube and the filled concrete. The predicted load versus deformation relationship was in good agreement with test results. The theoretical model was used to investigate the influence of important parameters that determine the ultimate strength of the composite columns. The parametric and experimental studies provide information for the development of formulae for the calculation of the ultimate strength and the axial load versus axial strain curves of the composite columns. Comparisons are made with predicted stub column strengths using the existing codes, such as ACI-1999, AISC-LRFD-1999, AIJ-1997, BS5400-1979 and EC4-1994.

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Nomenclature

A_s	Steel cross-sectional area
A_c	Concrete cross-sectional area
B	Width of square steel tube
b_e	Effective width of steel plate
CHS	Circular hollow section
D	Diameter of circular steel tube
DI	Ductility index
E_c	Concrete modulus of elasticity
E_s	Steel modulus of elasticity
f_{sy}	Yield strength of steel
f_{cu}	Concrete cube strength
f_{ck}	Characteristic concrete strength ($f_{ck} = 0.67 f_{cu}$ for normal strength concrete)
f'_c	Concrete cylinder strength
HSS	Hollow structural steel
N	Axial load
N_u	Axial compressive capacity
N_{uc}	Predicted ultimate strength
N_{ue}	Experimental ultimate strength
NC	Normal concrete
SCC	Self-consolidating concrete
SHS	Square hollow section
SI	Strength index
t	Wall thickness of steel tube
α	Steel ratio ($= A_s/A_c$)
σ	Stress
σ_{ol}	Local buckling stress
ε	Strain
ε_{ue}	Experimental ultimate strain
μ_s	Poisson's ratio of steel
ξ	Confinement factor ($\xi = \frac{A_s \cdot f_{sy}}{A_c \cdot f_{ck}}$)

Keywords: Composite columns; Composite action; Confinement factor; Concrete; Design; Hollow sections; Sectional capacity

1. Introduction

Self-consolidating concrete (SCC), as it is sometimes known, arrived as a revolution in the field of concrete technology whereby the concrete can flow under its own weight and fill in the formwork in the casting process. Due to its rheological properties, the expense of vibration is eliminated whilst still obtaining good consolidation. Apart from reliability and constructability, advantages of SCC include elimination of noise level in processing plants and reduction in construction time and labour cost. SCC has gained the attention of engineers as demonstrated by the First North American Conference on the Design and Use of Self-Consolidating Concrete held in Northwestern University, November 2002. There is a potential for using SCC-filled Hollow Structural Steel (HSS) as columns in structures.

Studies on the properties of SCC have been reported recently in the literature, for example Persson [16], Zhu and Peter [21]. Persson [16] found that the elastic modulus, creep and shrinkage of SCC did not differ significantly from the corresponding properties of normal concrete (NC). Zhu and Peter [21] studied the permeation properties of a range of SCC mixes. Their results indicated that the SCC mixes had significantly lower oxygen permeability than the vibrated normal concretes of the same strength grades.

In the past, there were a large number of research studies carried out on NC-filled HSS stub columns. This literature has been generally reviewed by Schneider [17]. For the purpose of “full” review on the literatures, Han [8] listed some of the review results and paragraphs presented in Schneider [17], and reviewed more references.

The present study is an attempt to study the behaviour of hollow structural steel (HSS) stub columns filled with SCC. The main objectives of this research were fourfold. First, to report a series of new tests on composite columns filled with SCC. Second, to analyse the axial load versus axial deformation behaviour for concrete-filled HSS stub columns based on the unified theory, which was presented by Han [9] and Han et al. [12]. Third, to develop a simplified model for calculating the sectional capacity and the axial load versus axial deformation relationships of the composite stub columns. And fourth, to compare the predicted ultimate strengths using the existing codes, such as ACI [1], AIJ [2], AISC-LRFD [3], BS5400 [5] and EC4 [7].

2. Experimental programme

2.1. Test specimens

A total of 50 specimens were tested. The main parameters varied in the tests are: (1) sectional types: circular and square; (2) steel yielding strength: from 282 to 404 MPa; and (3) tube diameter or width to wall thickness ratio (D/t or B/t): from 30 to 134.

A summary of the specimens is presented in Table 1. The lengths of stub columns (L) were chosen to be three times the diameter (for CHS) or the width (for SHS) to avoid the effects of overall buckling and end conditions [12].

The tubes were all manufactured from mild steel sheet, with the plate being cut from the sheet, tack welded into a circular or square shape and then welded with a single bevel butt weld.

Table 1

Specimen labels, material properties, section capacities and ductility index

Section type	No.	Specimen number	$D(B)$ (mm)	t (mm)	$D(B)/t$	f_{sy} (MPa)	f_{cu} (MPa)	N_{ue} (kN)	SI	$\epsilon_{ue}(\mu\epsilon)$	DI
Circular	1	CA1-1	60	1.87	32.1	282	85.2	312	1.429	9022	2.738
	2	CA1-2	60	1.87	32.1	282	85.2	320	1.361	8078	2.364
	3	CA2-1	100	1.87	53.5	282	85.2	822	1.368	6000	2.317
	4	CA2-2	100	1.87	53.5	282	85.2	845	1.407	5446	1.946
	5	CA3-1	150	1.87	80.2	282	85.2	1701	1.353	5300	1.763
	6	CA3-2	150	1.87	80.2	282	85.2	1670	1.329	5500	2.036
	7	CA4-1	200	1.87	107	282	85.2	2783	1.295	4800	2.052
	8	CA4-2	200	1.87	107	282	85.2	2824	1.314	4100	1.919
	9	CA5-1	250	1.87	133.7	282	85.2	3950	1.205	3800	2.002
	10	CA5-2	250	1.87	133.7	282	85.2	4102	1.251	5200	1.980
	11	CB1-1	60	2	30	404	85.2	427	1.445	8600	3.145
	12	CB1-2	60	2	30	404	85.2	415	1.404	10700	2.804
	13	CB2-1	100	2	50	404	85.2	930	1.359	5000	1.762
	14	CB2-2	100	2	50	404	85.2	920	1.344	7498	2.227
	15	CB3-1	150	2	75	404	85.2	1870	1.352	5000	2.120
	16	CB3-2	150	2	75	404	85.2	1743	1.260	6500	2.220
	17	CB4-1	200	2	100	404	85.2	3020	1.302	4639	1.896
	18	CB4-2	200	2	100	404	85.2	3011	1.298	3775	2.030
	19	CB5-1	250	2	125	404	85.2	4442	1.272	4100	1.876
	20	CB5-2	250	2	125	404	85.2	4550	1.303	4001	2.150
	21	CC1-1	60	2	30	404	90	432	1.417	8531	2.661
	22	CC1-2	60	2	30	404	90	437	1.433	8600	2.697
	23	CC2-1	150	2	75	404	90	1980	1.368	4700	1.978
	24	CC2-2	150	2	75	404	90	1910	1.320	4583	2.188
	25	CC3-1	250	2	125	404	90	4720	1.286	3600	2.175
	26	CC3-2	250	2	125	404	90	4800	1.307	3820	2.165
Square	27	SA1-1	60	1.87	32.1	282	81	382	1.263	4600	2.743
	28	SA1-2	60	1.87	32.1	282	81	350	1.157	4400	2.671
	29	SA2-1	100	1.87	53.5	282	81	860	1.173	3374	2.667
	30	SA2-2	100	1.87	53.5	282	81	840	1.146	3100	2.410
	31	SA3-1	150	1.87	80.2	282	81	1662	1.088	1700	2.529
	32	SA3-2	150	1.87	80.2	282	81	1740	1.139	2600	2.277
	33	SA4-1	200	1.87	107	282	81	2890	1.109	1200	2.358
	34	SA4-2	200	1.87	107	282	81	2920	1.121	2490	2.033
	35	SA5-1	250	1.87	133.7	282	60	3304	1.098	1477	1.978
	36	SA5-2	250	1.87	133.7	282	60	3400	1.130	1550	2.256
	37	SB1-1	60	2	30	404	50.9	318	1.179	4230	2.576
	38	SB1-2	60	2	30	404	50.9	322	1.193	5778	2.356
	39	SB2-1	100	2	50	404	50.9	770	1.219	4000	2.225
	40	SB2-2	100	2	50	404	50.9	772	1.222	3400	2.475
	41	SB3-1	150	2	75	404	50.9	1300	1.077	2200	2.350
	42	SB3-2	150	2	75	404	50.9	1420	1.176	2400	1.975
	43	SB4-1	200	2	100	404	50.9	1990	1.019	1400	2.465
	44	SB4-2	200	2	100	404	50.9	2054	1.051	2289	2.204
	45	SB5-1	250	2	125	404	50.9	3100	1.080	2000	2.350
	46	SB5-2	250	2	125	404	50.9	2965	1.033	1496	2.105
	47	SC1-1	60	2	30	404	81	422	1.154	5231	2.373
	48	SC1-2	60	2	30	404	81	406	1.111	4800	2.477
	49	SC2-1	150	2	75	404	81	2060	1.220	4000	2.225
	50	SC2-2	150	2	75	404	81	1980	1.172	3500	2.195

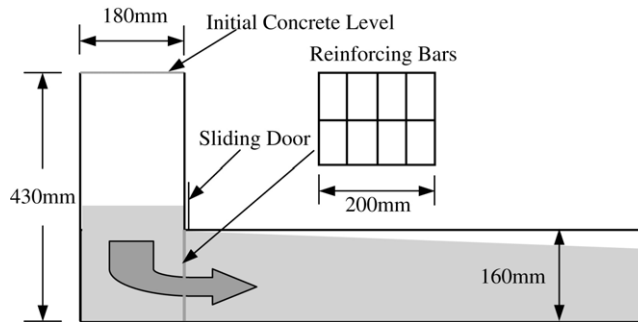


Fig. 1. A schematic view of the L-box.

Two types of steel were used. Strips of the steel tubes were tested in tension. Three coupons were taken from each face of the steel tube; from these tests, the average yield strength (f_{sy}) of the tube was found to be 282 and 404 MPa. The ultimate strength was found to be 358 and 514 MPa, and the modulus of elasticity was about 201,500 MPa and 207,000 MPa respectively.

The SCC mix was designed for compressive cube strength (f_{cu}) at 28 days of approximately 60 MPa. The modulus of elasticity (E_c) of concrete was measured, the average value being 41,540 MPa. The mix proportions were as follows:

- Cement: 400 kg/m³.
- Blast furnace slag: 170 kg/m³.
- Water: 181 kg/m³.
- Sand: 815 kg/m³.
- Coarse aggregate: 815 kg/m³.
- Additional high-range water reducer (HRWR): 5.2 kg/m³.

In all the concrete mixes, the fine aggregate used was silica-based sand, while the coarse aggregate was carbonate stone from Fuzhou City, South China.

A typical test set-up (L-Box) for simulating the casting process by forcing an SCC sample to flow through a reinforcement net under a static pressure, in China, is shown in Fig. 1. The flow time from the sliding door to the front door of the L-box, the flow speed, as well as the flow distance of the SCC were recorded. The fresh properties of the SCC mixture were as follows:

- Slump flow (mm): 268.
- Fresh air content (%): 4.5.
- Unit weight (kg/m³): 2386.
- Concrete temperature (°C): 26.
- Flow time (s): 14.
- Flow speed (mm/s): 57.
- Flow distance (mm): 800.

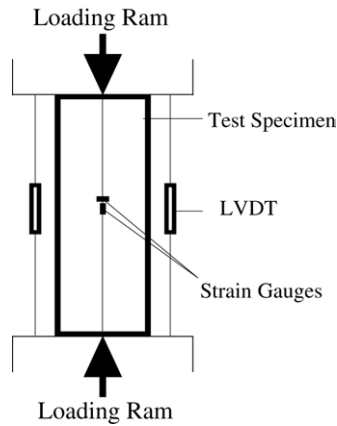


Fig. 2. Arrangement of stub column tests.

The average cube strengths of each specimen at the time of the test are listed in Table 1. The self-consolidating concrete was cured without any vibration.

For casting the test specimens, the ends of the steel tubes were cut and machined to the required length. The insides of the tubes were wire brushed to remove any rust and loose debris present. The deposits of grease and oil, if any, were cleaned away. Each tube was welded to a circular (for circular sections) or a square (for square sections) steel base plate 10 mm thick. The specimens were placed upright to air-dry until testing occurred. During curing, a very small amount of longitudinal shrinkage of 0.2 mm or so occurred at the top of the column. A high-strength epoxy was used to fill this longitudinal gap so that the concrete surface was flush with the steel tube at the top.

2.2. Test set-up

The experimental study was to determine not only the maximum load carrying capacity of the specimens, but also to investigate the failure pattern up to and beyond the ultimate load.

All the tests were performed on a 5000 kN capacity testing machine. The specimens were placed into the testing machine and the loads were applied on the specimens directly. Fig. 2 gives a schematic view of the test arrangement. The loading ram is a solid steel plate, which acts like an end stiffener. Eight strain gauges were used for each specimen to measure strains at the mid-height. Two linear voltage–displacement transducers (LVDTs) were used to measure the axial deformation, as shown in Fig. 2. A load interval of less than one tenth of the estimated carrying load capacity was used. Each load interval was maintained for about 2 min.

2.3. Failure modes and load–axial strain curves

A typical failure mode of the outer tube was the local (outward folding) failure mechanism; the failure modes of the stub columns with different sections are shown in

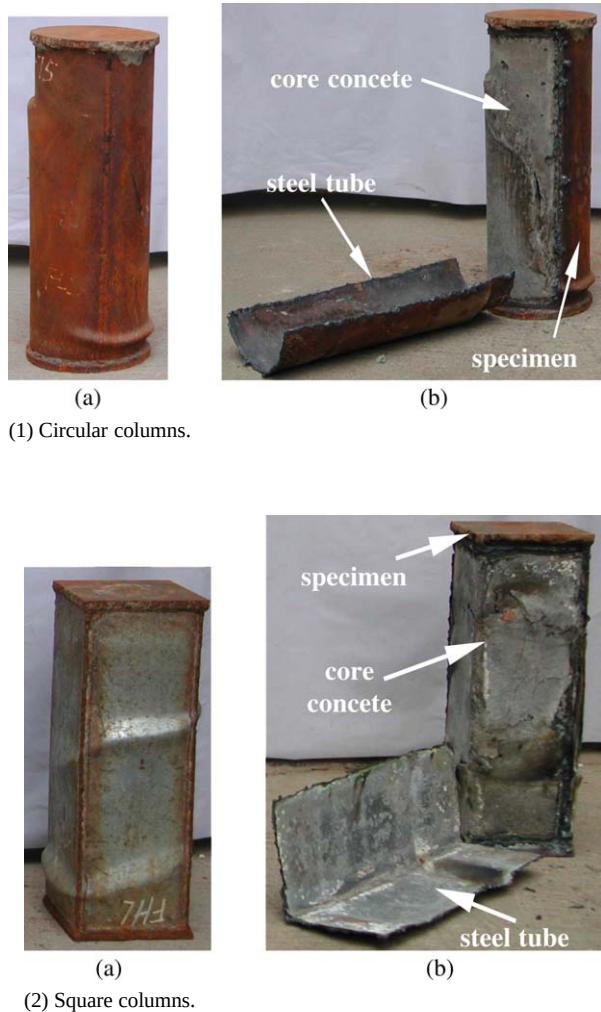


Fig. 3. Stub column failure mode.

Fig. 3. This is the same as that observed by many other researchers for normal concrete-filled steel tubes, such as Uy [19] Han et al. [12], O'Shea and Bridge [13–15].

All the recorded curves of load (N) versus axial strain (ϵ) are shown in Figs. 4 and 5 for specimens with circular and square sections respectively. The maximum loads (N_{ue}) and the ultimate strain (ϵ_{ue}) corresponding to N_{ue} obtained in the test are summarized in Table 1. Figs. 6 and 7 show the measured curves of load (N) versus lateral strain (ϵ) for specimens with circular and square sections respectively.

All tests show similar behaviour of axial load (N) versus axial strain (ϵ) relationships, shown in Fig. 8.

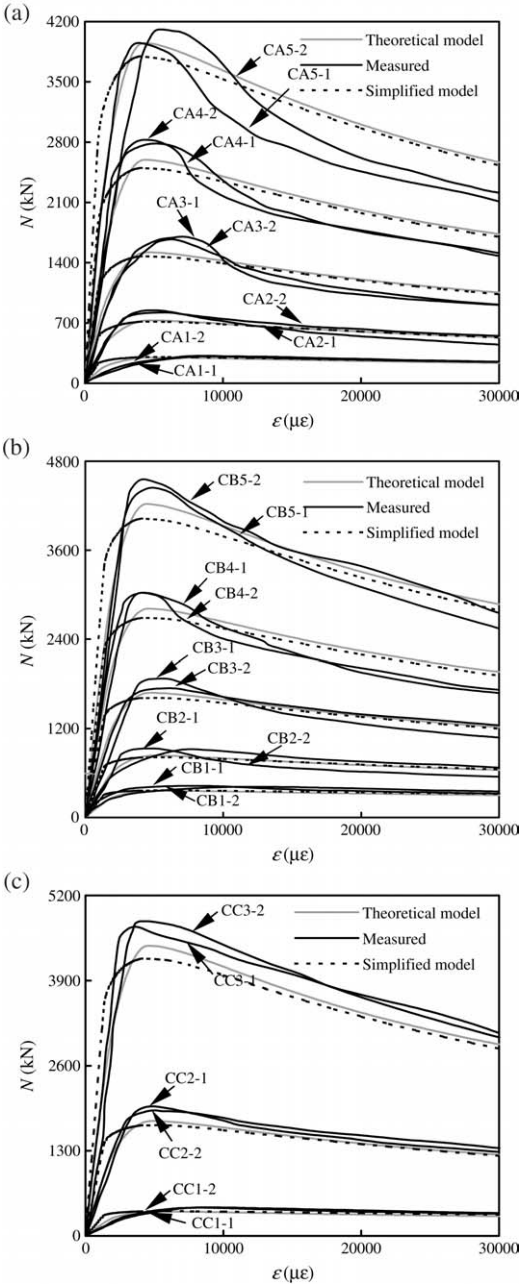


Fig. 4. Axial load (N) versus axial strain (ϵ) curves (circular sections).

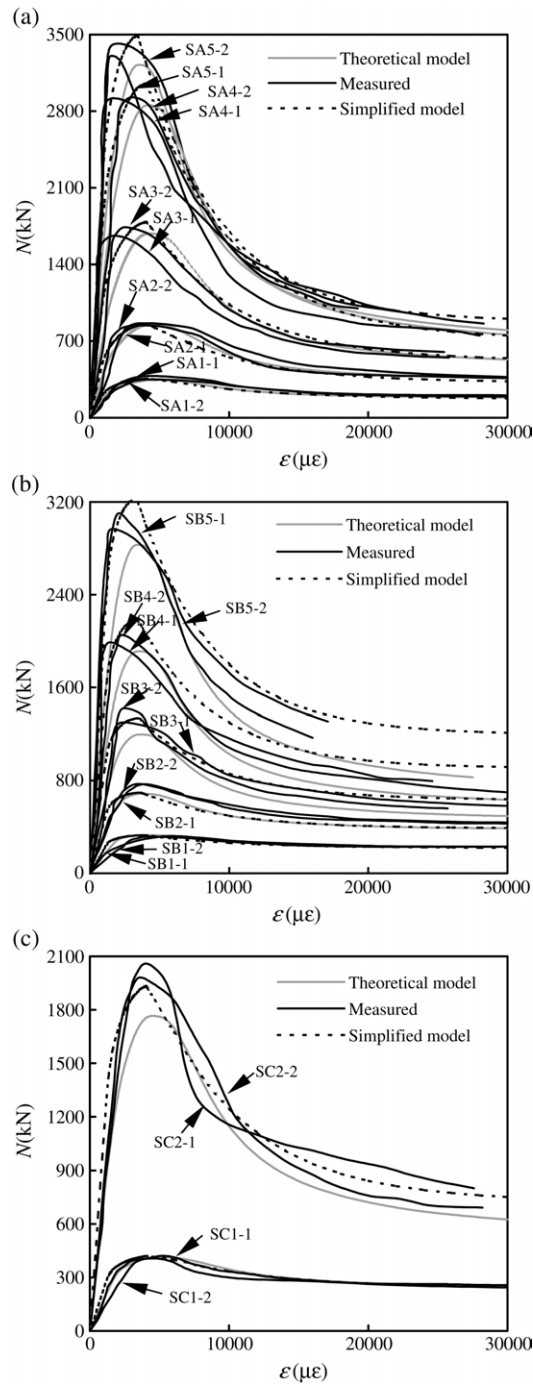


Fig. 5. Axial load (N) versus axial strain (ϵ) curves (square sections).

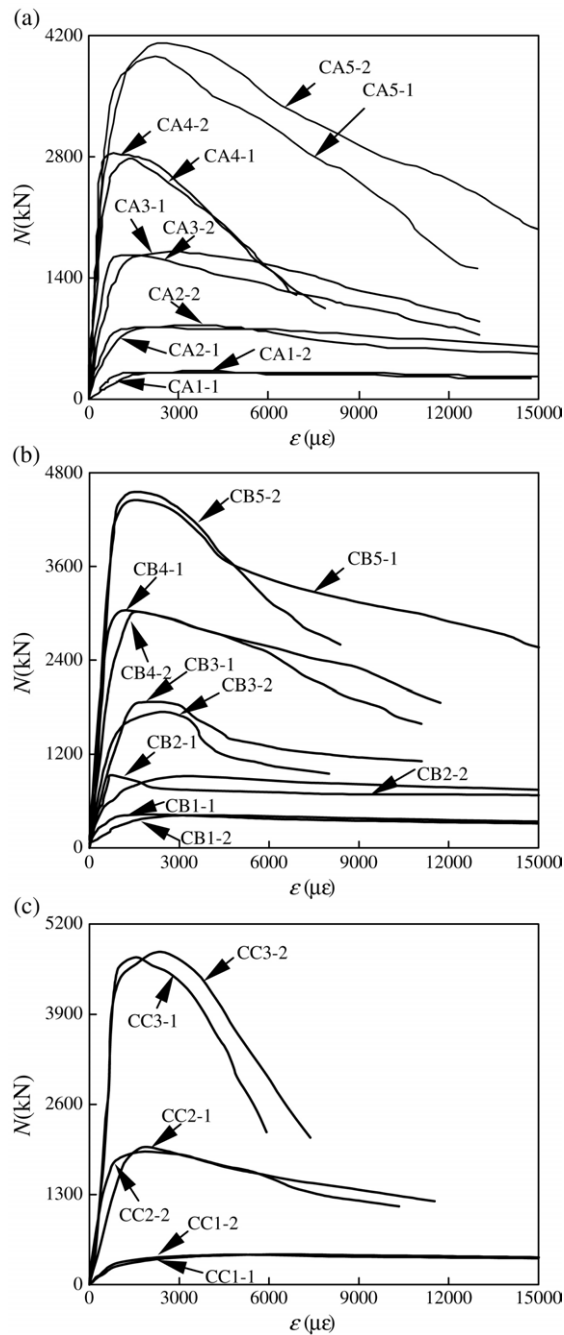


Fig. 6. Axial load (N) versus lateral strain (ϵ) curves (circular sections).

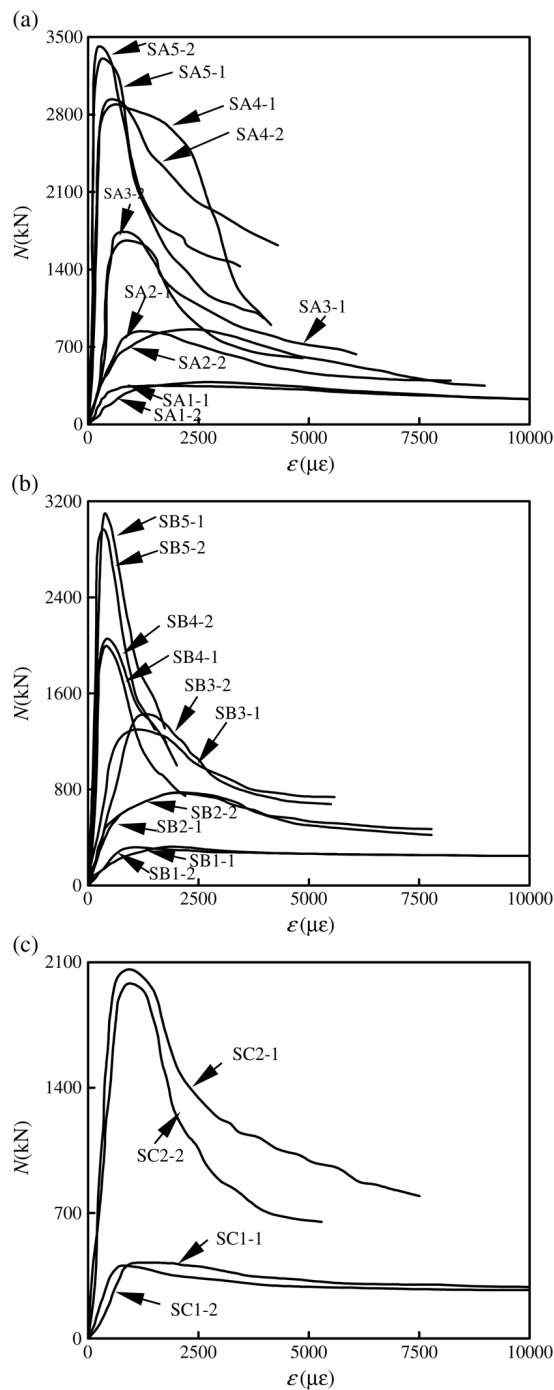


Fig. 7. Axial load (N) versus lateral strain (ϵ) curves (square sections).

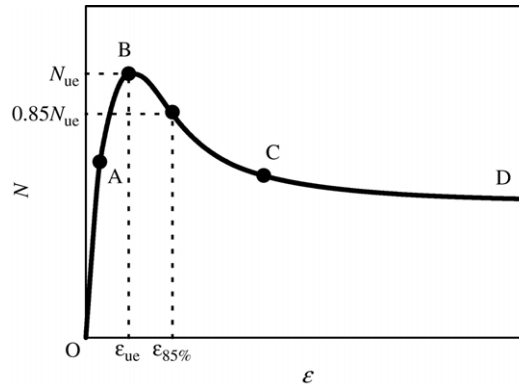


Fig. 8. A schematic view of the measured axial load (N) versus axial strain (ε) curves.

2.4. Strength index

A strength index is defined to quantify the section strength:

$$SI = \frac{N_{ue}}{N_{uo}} \quad (1)$$

where $N_{uo} = A_s \cdot f_y + 0.85A_c \cdot f'_c$, giving the sectional capacity as in ACI [1].

The strength indexes (SI) so determined are listed in Table 1. They are plotted in Fig. 9(a) and (b) against the tube diameter or the width to wall thickness ratio (D/t or B/t) for the specimens with circular and square sections respectively. It can be seen from Fig. 9 that generally, the strength index decreases with increase of D/t or B/t . The reasons are that the constraining factor ($\xi = \frac{A_s \cdot f_{sy}}{A_c \cdot f_{ck}}$) for the specimens decreases with increase of D/t or B/t , i.e. the composite action of steel tube and core concrete becomes smaller.

2.5. Ductility index

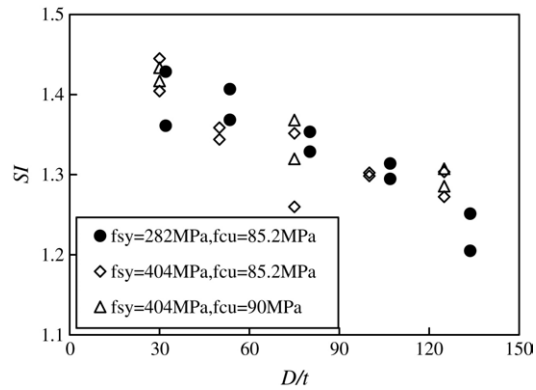
One of the parameters used to quantify section ductility is the ductility index. The definition given in [8] is adopted in this paper. It is expressed as

$$DI = \frac{\varepsilon_{85\%}}{\varepsilon_{ue}} \quad (2)$$

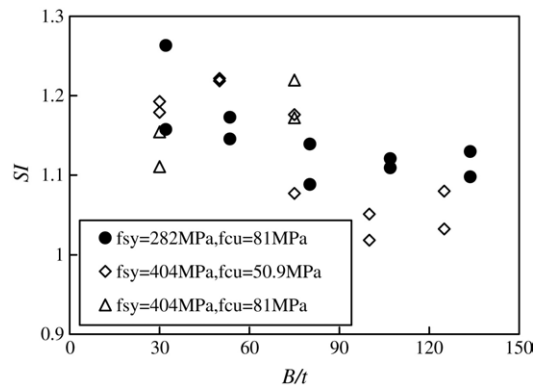
where, as shown in Fig. 10, ε_{ue} is the strain at the ultimate load, and $\varepsilon_{85\%}$ is the strain when the load falls to 85% of the ultimate load.

The ductility indexes (DI) so determined are listed in Table 1. They are plotted in Fig. 10(a) and (b) against the tube diameter or width to wall thickness ratio (D/t or B/t) for the specimens with circular and square sections respectively. It can be seen from Fig. 10 that, in general, the ductility index decreases with increase in D/t or B/t . The reasons are very similar to the causes of the strength index changing with increase of D/t or B/t .

At the same time, it can be seen from Figs. 9 and 10 that generally a greater reduction for SHS occurs. The reasons are that the constraining effect for CHS is better than for SHS with the same constraining factor (ξ) and SHS are more prone to local buckling than CHS.



(a) Circular sections.



(b) Square sections.

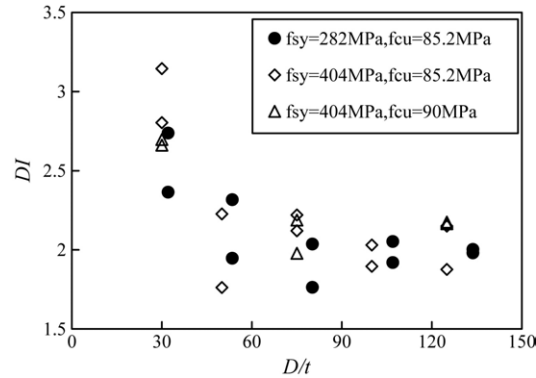
Fig. 9. SI versus $D(B)/t$ relations.

3. Mechanics model

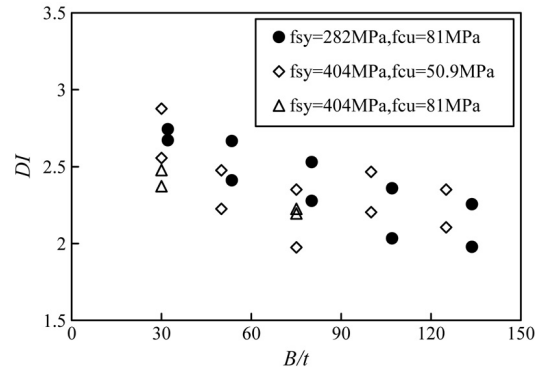
A numerical model was developed for the analysis of the behaviour of concrete-filled HSS stub columns. In the analysis, the effects of local buckling and residual stress are considered, and this was an extension of the former model, which was presented by Han et al. [12].

In this method, for the calculation of the load versus deformation relations of the stub composite columns, the following assumptions were made:

- (1) There is no slip between the steel and concrete.
- (2) The stress–strain relationships for steel and for the confined concrete given by Han et al. [12], Han and Huo [10] are adopted. Details of the stress–strain relationship for the confined concrete based on NC are summarized in [Appendix A](#) in this paper.
- (3) Residual stress distributions for a steel plate of a column shown as in [Fig. 11](#) is used in the analysis [11].



(a) Circular sections.



(b) Square sections.

Fig. 10. DI versus $D(B)/t$ relations.

- (4) An effective width model for square sections, illustrated in Fig. 12, was adopted [18]. The effective width of b_e can be given by

$$\frac{b_e}{b} = \alpha \cdot \sqrt{\frac{f_{ol}}{f_{sy}}} \quad (3)$$

where $\alpha = 0.65$ according to the Australian Standard AS4100 [4]; The buckling stress is calculated using the well-known equation of Bryan [6], i.e. $f_{ol} = \frac{\pi^2 E_s}{12(1-\mu_s^2)} \frac{k}{(b/t)^2}$; E_s and μ_s are the modulus and the Poisson's ratio of the steel respectively; the local buckling coefficient k is taken as 10.3 [20].

- (5) Force equilibrium and deformation consistencies are considered along the longitudinal direction, i.e.

$$N = N_s + N_c \quad (4)$$

$$\varepsilon_{sl} = \varepsilon_{cl} \quad (5)$$

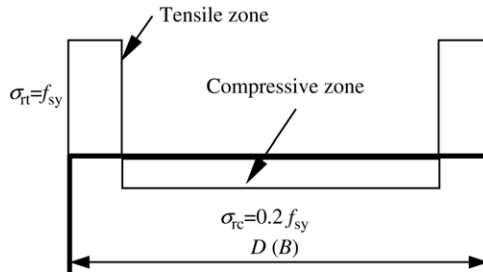


Fig. 11. Residual stress distributions.

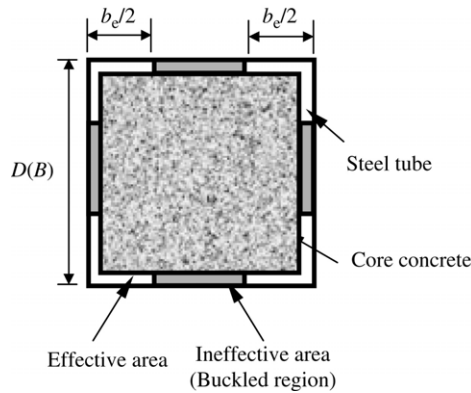


Fig. 12. Effective width of concrete-filled steel SHS sections.

in which N_s and N_c are forces carried by steel and concrete, ε_{sl} and ε_{cl} are longitudinal strains in steel and concrete.

The procedures for calculating load versus axial strain are expressed as follows:

For a given (i th) increment in axial strain $d\varepsilon_{li} \rightarrow i$ th axial strain $\varepsilon_{l,i+1} = \varepsilon_{li} + d\varepsilon_{li} \rightarrow i$ th axial stress $\sigma_{sl,i+1}$ and $\sigma_{cl,i+1} \rightarrow i$ th forces $N_{s,i}$ and $N_{c,i} \rightarrow i$ th force N_i or combined stress $\sigma_{sc} = N_i / A_{sc}$, where $A_{sc} = A_s + A_c$.

Typical curves of combined stress versus axial strain are shown schematically in Fig. 13. It can be seen that the curves are a function of the confinement factor (ξ). There are three or four stages in the load versus axial curves, i.e.

Stage 1: Elastic stage (from point O to point A). During this stage, steel and concrete bear load independently. The yielding of steel occurs at point A.

Stage 2: Elastic–plastic stage (from point A to point B). During this stage, concrete is confined by the steel tube because the Poisson's ratio of the concrete is larger than that of steel. The confinement enhances as the longitudinal deformation increases.

Stage 3: Strain hardening stage (from point B to point C or from point B to point D). During this stage, the load versus axial strain curves tend to go upwards. The shape of the curve depends on the value of the confinement factor (ξ). When ξ is larger than ξ_0 , the curve goes up steadily to point C. When ξ is less than ξ_0 , the curve starts to go down after

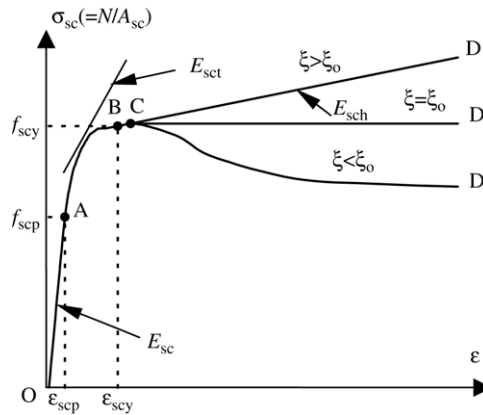


Fig. 13. Typical axial load (σ_{sc}) versus axial strain (ϵ) curves.

a short increase to point C. The smaller the confinement factor, the earlier the curve starts to fall down. It was found that the values of ξ_0 for concrete-filled steel CHS and SHS can be given as 1.1 and 4.5 respectively [9].

Stage 4: Falling stage (from point C to point D). This occurs only when the confinement factor (ξ) is less than ξ_0 . The smaller the confinement factor, the steeper the curve fall.

The predicted curves are compared with the measured curves for the current tests in Figs. 4 and 5. It can be seen that reasonable agreement is achieved.

The column section capacities predicted using the mechanics model are compared with the current circular stub column tests. Fig. 14(a) shows the comparisons, where a mean of 0.915 and a COV (coefficient of variation) of 0.038 are obtained. Fig. 14(b) shows the comparisons between column sectional capacity predicted using the mechanics model with the current square stub column test results, where a mean of 0.952 and a COV of 0.054 are obtained.

4. Simplified model

4.1. Section capacity

Fig. 15(1) and (2) show the relationships between axial stress ($\sigma_{sc} = N/A_{sc}$) and axial strain (ϵ) of the composite columns with circular and square sections under different conditions respectively. It can be concluded that the σ_{sc} – ϵ relationships show strain hardening or an elastic–perfectly plastic behaviour with larger confinement factor (ξ), while for columns with smaller ξ , the σ_{sc} – ϵ curves are of degrading types.

The load corresponding to the longitudinal strain ϵ_{scy} was chosen as the ultimate strength of the stub columns. Using the procedure in Fig. 13 for the three different types of load–strain curves, Han [9] has determined the value of ϵ_{scy} as follows:

For concrete-filled steel CHS sections:

$$\epsilon_{scy} = 1300 + 12.5 f'_c + (600 + 33.3 f'_c) \cdot \xi^{0.2}. \quad (6a)$$

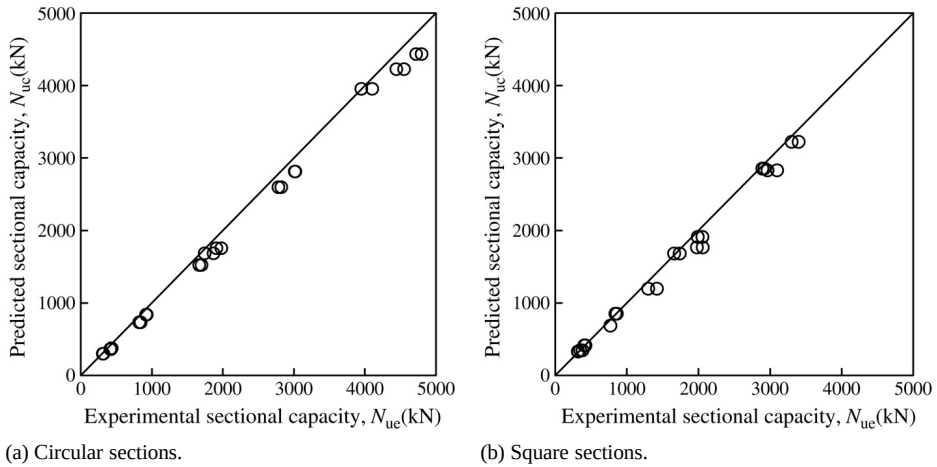


Fig. 14. Comparison of sectional capacity between theoretical model and tests.

For concrete-filled steel SHS sections:

$$\varepsilon_{scy} = 1300 + 12.5 f'_c + (570 + 31.7 f'_c) \cdot \xi^{0.2}. \quad (6b)$$

The considerations for the determination of ε_{scy} can be summarized as follows:

- (1) ε_{scy} is the strain corresponding to the peak load, for the σ_{sc} – ε curves are of degrading types. While the σ_{sc} – ε curves showed strain hardening or an elastic–perfectly plastic behaviour, the elastic–plastic stage, the σ_{sc} – ε curves generally complete at the strain of ε_{scy} (point B in Fig. 13).
- (2) The steel reaches the yield strength f_{sy} , whilst the concrete reaches its ultimate compressive strength σ_o (see Appendix A) at point B.
- (3) The axial stress σ_{sc} increases relatively quickly with increase of axial strain when $\varepsilon < \varepsilon_{scy}$, while the axial stress σ_{sc} increases relatively slowly, or even decreases with increase of axial strain when $\varepsilon > \varepsilon_{scy}$ (see Fig. 13).

It was found that, within the parameter range of $f_{sy} = 200$ – 700 MPa, $f_{cu} = 30$ – 120 MPa and $\alpha = 0.03$ – 0.2 , ε_{scy} ranges from 2740 to 5200 $\mu\varepsilon$ for concrete-filled steel CHS columns, whilst ε_{scy} ranges from 2680 to 5050 $\mu\varepsilon$ for concrete-filled steel SHS columns.

For convenience of analysis, the “nominal yielding strength” of the composite sections (f_{scy}) is defined as follows:

$$f_{scy} = \frac{N_{uo}}{A_{sc}} \quad (7)$$

in which

N_{uo} – sectional capacity of the composite column;

A_{sc} – cross-section area of the composite section.

The compressive strength ratio of the composite sections ($\gamma_c = f_{scy}/f_{ck}$) so determined according to the mechanics model in this paper is plotted in Fig. 16 against the confinement

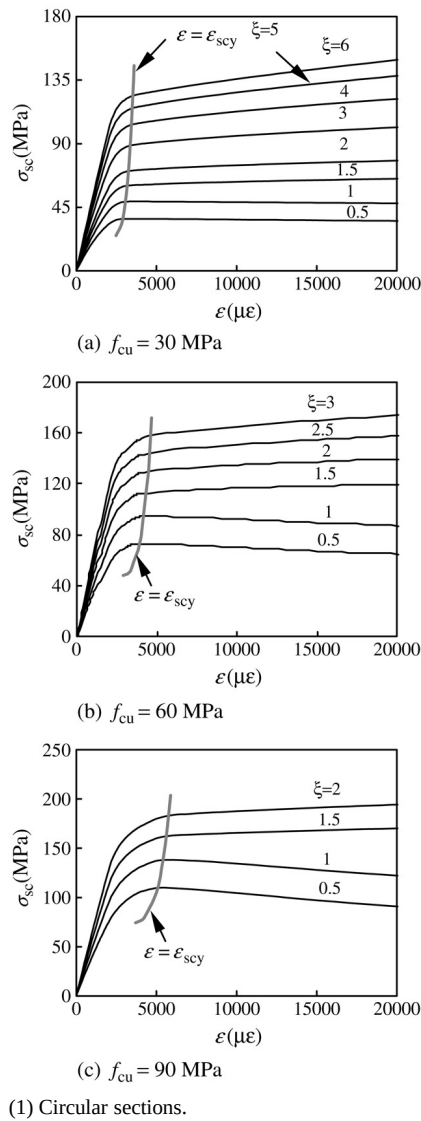
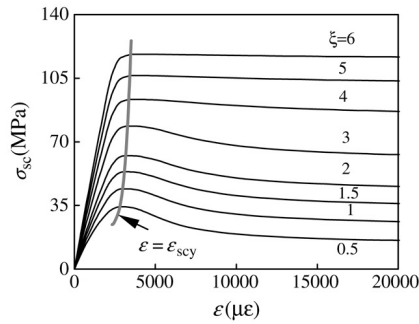
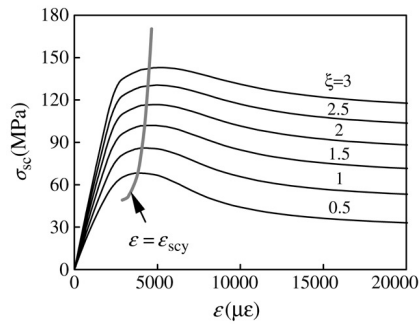
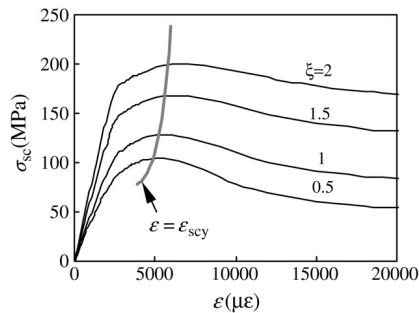


Fig. 15. σ_{sc} versus ε relations.

factor (ξ). It can be seen from Fig. 16 that the ratio $\gamma_c = f_{scy}/f_{ck}$ increases when the confinement factor (ξ) increases. Formulae for f_{scy} can be obtained by using the regression analysis method, i.e.

For concrete-filled steel CHS columns:

$$f_{scy} = (1.14 + 1.02\xi) \cdot f_{ck} \tag{8a}$$

(a) $f_{cu} = 30$ MPa(b) $f_{cu} = 60$ MPa(c) $f_{cu} = 90$ MPa

(2) Square sections.

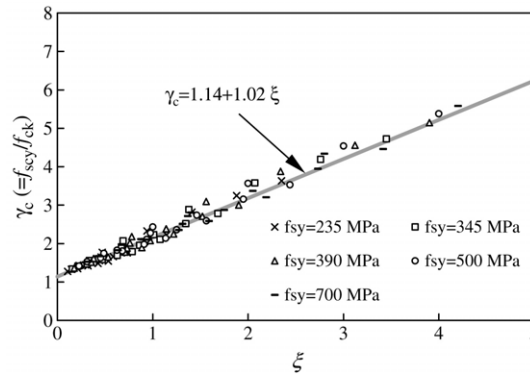
Fig. 15. (continued).

For concrete-filled steel SHS columns:

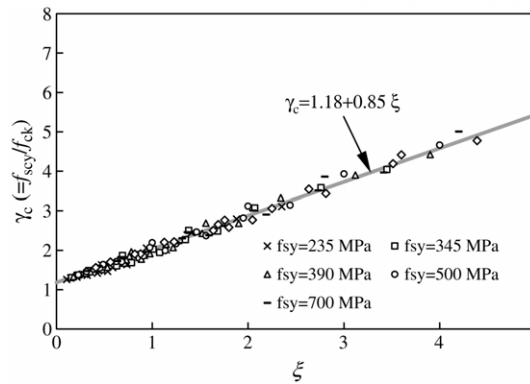
$$f_{scy} = (1.18 + 0.85\xi) \cdot f_{ck} \quad (8b)$$

The validity limit of Eq. (8) is: $0.1 < \xi < 5$.

It can be found that Eq. (8) is easier to use in calculations compared with the former simplified model, which was presented by Han et al. [12].



(a) Concrete-filled steel CHS.



(b) Concrete-filled steel SHS.

Fig. 16. $\gamma_c (= f_{scy}/f_{ck})$ versus ξ relations.

The concrete-filled steel CHS column section capacity predicted using formula (7) are compared with the current test results in Fig. 17(a) and (b) for columns with circular and square sections respectively, where a mean value of 0.888 and a COV (coefficient of variation) of 0.033 for circular columns, and a mean value of 1.011 and a COV of 0.055 for square columns are obtained.

4.2. Axial load versus axial deformation relations

On the basis of the numerical model mentioned above, according to the regression analysis method, the σ_{sc} – ε relations shown in Fig. 13 can be expressed as follows:

(1) Stage OA ($0 < \varepsilon \leq \varepsilon_{scp}$):

$$\sigma = E_{sc} \cdot \varepsilon \quad (9)$$

where E_{sc} is the elastic modulus of the composite sections, and can be given as

$$E_{sc} = f_{scp}/\varepsilon_{scp} \quad (10)$$

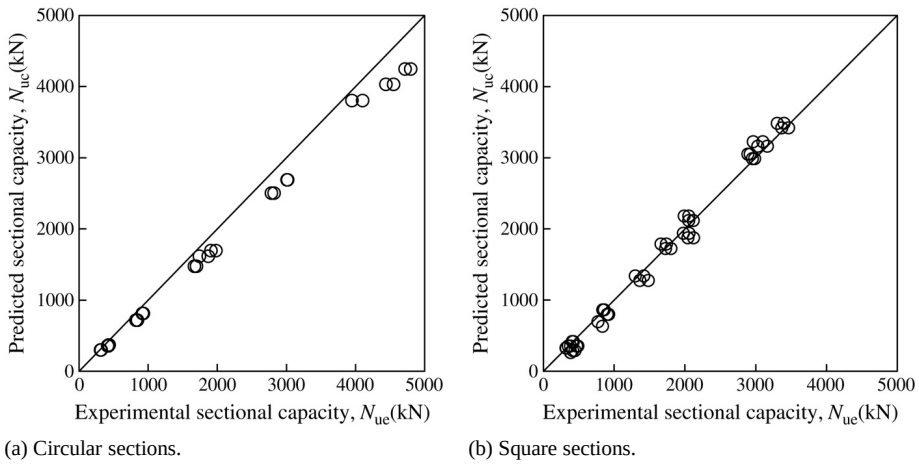


Fig. 17. Comparison of sectional capacity between the simplified model and test.

f_{scp} and ε_{scp} are the stress and strain corresponding to point A in Fig. 13, and can be given as:

(1) For concrete-filled steel CHS sections:

$$f_{scp} = [0.192(f_{sy}/235) + 0.488] \cdot f_{scy} \quad (11)$$

$$\varepsilon_{scp} = 3.25 \times 10^{-6} f_{sy}. \quad (12)$$

(2) For concrete-filled steel SHS sections:

$$f_{scp} = [0.263 \cdot (f_{sy}/235) + 0.365 \cdot (30/f_{cu}) + 0.104] \cdot f_{scy} \quad (13)$$

$$\varepsilon_{scp} = 3.01 \times 10^{-6} f_{sy} \quad (14)$$

where the unit for f_{sy} and f_{cu} is N/mm^2 .

(2) Stage AB ($\varepsilon_{scp} < \varepsilon \leq \varepsilon_{scy}$):

In this stage, $\sigma_{sc} - \varepsilon$ can be given as

$$\varepsilon^2 + a \cdot \sigma^2 + b \cdot \varepsilon + c \cdot \sigma + d = 0 \quad (15)$$

in which

$$a = \frac{-(\varepsilon_{scy} - \varepsilon_{scp})^2 - 2e \cdot (\varepsilon_{scy} - \varepsilon_{scp})}{(f_{scy}^2 - f_{scp}^2) + (\varepsilon_{scy} - \varepsilon_{scp}) \cdot (-2f_{scp} \cdot E_{sc}) + e \cdot (2f_{scy} \cdot k - 2f_{scp} \cdot E_{sc})};$$

$$b = -2\varepsilon_{scp} - 2a \cdot f_{scp} \cdot E_{sc} - c \cdot E_{sc};$$

$$c = \frac{2(\varepsilon_{scy} - \varepsilon_{scp}) + (2f_{scy} \cdot k - 2f_{scp} \cdot E_{sc}) \cdot a}{E_{sc} - k};$$

$$d = -\varepsilon_{scp}^2 - a \cdot f_{scp}^2 - b \cdot \varepsilon_{scp} - c \cdot f_{scp};$$

$$e = \frac{(f_{scy} - f_{scp}) - E_{sc} \cdot (\varepsilon_{scy} - \varepsilon_{scp})}{E_{sc} - k}.$$

Here, when $\xi < \xi_0$, $k = 0$; when $\xi \geq \xi_0$, $k = E_{sch}$, where E_{sch} is the tangent modulus in the hardening stage. For concrete-filled steel CHS columns, $\xi_0 = 1.1$, $E_{sch} = 270\xi - 100$. For concrete-filled steel SHS columns, $\xi_0 = 4.5$, $E_{sch} = 30\xi - 40$.

(3) Hardening or descending stage BD ($\varepsilon > \varepsilon_{scy}$):

For concrete-filled steel CHS sections:

$$\sigma = \begin{cases} f_{scy} + E_{sch}(\varepsilon - \varepsilon_{scy}) & (\xi \geq 1.1) \\ \frac{f_{scy} \cdot \varepsilon}{D(\varepsilon - \varepsilon_{scy})^2 + \varepsilon} & (\xi < 1.1) \end{cases} \quad (16a)$$

in which

$$\begin{aligned} D &= p \cdot \xi^2 + q \cdot \xi + r; \\ p &= 0.52\beta_c^2 + 5.6\beta_c - 5.85; \\ q &= -1.36\beta_c^2 - 10.2\beta_c + 7.3; \\ r &= 0.44\beta_c^2 + 9.4\beta_c - 4.8; \\ \beta_c &= f_{cu}/30, \end{aligned}$$

where the unit for f_{cu} is N/mm².

For concrete-filled steel SHS sections:

$$\sigma = \begin{cases} f_{scy} + E_{sch}(\varepsilon - \varepsilon_{scy}) & (\xi \geq 4.5) \\ f_{scy} \cdot \left(1 - \beta_\xi + \beta_\xi \cdot \exp - \left(\frac{\varepsilon - \varepsilon_{scy}}{m} \right) \right) & (\xi < 4.5) \end{cases} \quad (16b)$$

in which

$$\begin{aligned} \beta_\xi &= -0.194 \ln \xi + 0.445; \\ m &= (n \cdot \xi + l) \times 10^{-3}; \\ n &= 0.5\beta_c + 0.18; \\ l &= 0.8\beta_c + 3.74; \\ \beta_c &= f_{cu}/30, \end{aligned}$$

where the unit for f_{cu} is N/mm².

Figs. 4 and 5 compare the calculated N – ε relation curves among the tests, the mechanics model and the simplified model. Comparisons of σ_{sc} – ε relation curves between the mechanics model and the simplified model are shown in Fig. 18; it can be seen that good agreement is achieved.

5. Comparisons with current code provisions

The sectional capacities predicted using the following six design methods are compared with the stub column test results:

- ACI [1].
- AIJ [2].
- AISC-LRFD [3].
- BS5400 [5].
- EC4 [7].

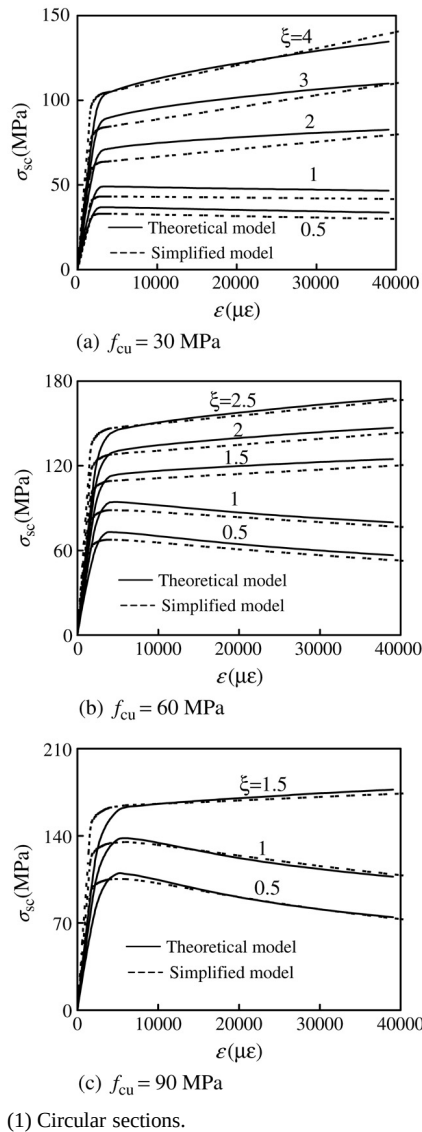
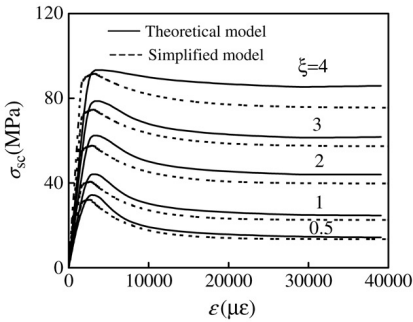


Fig. 18. Comparison of predicted σ_{sc} versus ϵ curves between the theoretical model and simplified model.

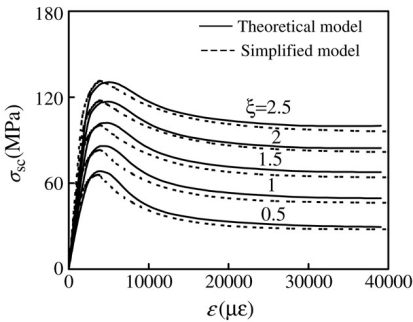
In all the design calculations, the material partial safety factors were set to unity.

Predicted section capacities (N_{uc}) using the different methods are compared with test results (N_{ue}). For overall comparisons, Table 2 shows both the mean value (μ) and the coefficient of variation (COV) of the ratio of N_{uc}/N_{ue} for the different design methods.

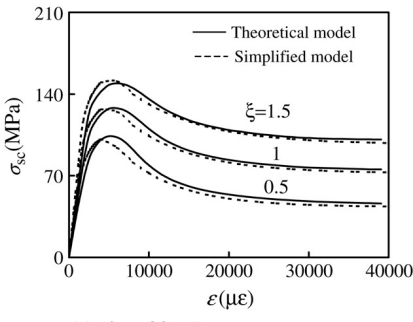
Results in Table 2 show that ACI [1], AIJ [2], AISC-LRFD [3] and BS5400 [5] are conservative for predicting the sectional capacities of the specimens with circular sections.



(a) $f_{cu} = 30$ MPa



(b) $f_{cu} = 60$ MPa



(c) $f_{cu} = 90$ MPa

(2) Square sections.

Fig. 18. (continued).

Table 2
Comparison between predicted section capacities (N_{uc}) and test results (N_{ue})

Section type	N_{uc}/N_{ue}									
	ACI		AIJ		AISC-LRFD		BS5400		EC4	
	(1999)		(1997)		(1999)		(1979)		(1994)	
	μ	COV	μ	COV	μ	COV	μ	COV	μ	COV
Circular	0.788	0.039	0.843	0.028	0.785	0.041	0.952	0.054	1.006	0.049
Square	0.919	0.050	0.919	0.050	0.916	0.052	0.878	0.054	1.027	0.067

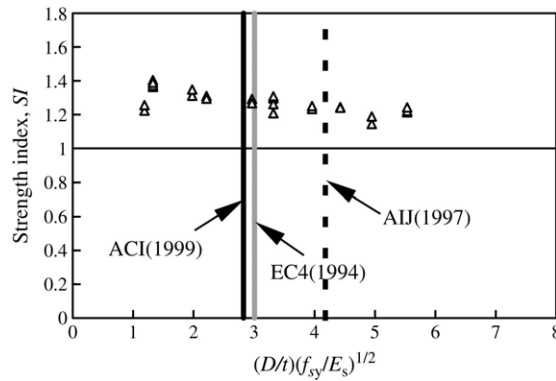


Fig. 19. Strength index (SI) versus $(D/t) \cdot \sqrt{f_{sy}/E_s}$ relations (circular columns).

Overall, ACI [1] and AISC-LRFD [3] gave a sectional capacity about 20% lower than the results obtained in the tests. AIJ [2] and BS5400 [5] gave a section capacity about 16% and 5% lower than the measured ultimate strengths respectively. Overall, the methods proposed by EC4 [7] and BS5400 [5] give means of 1.006 and 0.952, and COVs of 0.049 and 0.054 respectively, are the best means predicted.

Results in Table 2 also clearly show that ACI [1], AIJ [2], AISC-LRFD [3] and BS5400 [5] are conservative for predicting the sectional capacities of the specimens with square sections. Overall, the four codes gave a sectional capacity about 10% lower than the results obtained in the tests. However, EC4 [7] gave a section capacity about 3% higher than these of the measured ultimate strength, and gave an unsafe prediction. Overall, the methods proposed by ACI [1] and AIJ [2] give a mean of 0.919 and a COV of 0.050, is the best mean predicted.

The existing codes (ACI-1999, AISC-LRFD-1999, AIJ-1997, BS5400-1979 and EC4-1994) were calibrated only for NC-filled members. They are used in this paper to predict the section capacity (N_{uc}) of SCC-filled HSS stub columns. It seems that the conclusion regarding predictions using existing design codes made for NC-filled HSS stub columns remains the same for SCC-filled HSS stub columns.

6. Discussions on the upper limits of the D/t and B/t ratios

The limiting values of the diameter-to-thickness ratio for a circular tube and the width-to-thickness ratio for a square tube are given in the different codes. The current stub column test results were used to examine the limiting diameter (width)-to-thickness ratio for the composite columns.

Figs. 19 and 20 show the strength index (SI) versus diameter (width)-to-thickness ratios relations for circular and square sections, respectively, where the strength index (SI) is calculated from Eq. (1). The limiting values of the diameter-to-thickness ratio for the composite sections, given in ACI [1], AIJ [2] and EC4 [7], were also given in Figs. 19 and 20.

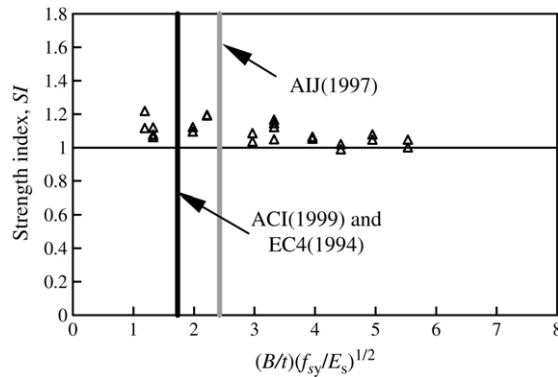


Fig. 20. Strength index (SI) versus $(B/t) \cdot \sqrt{f_{sy}/E_s}$ relations (square sections).

The following observations can be made from Figs. 19 and 20: in general, the values of SI obtained for both circular and square specimens become smaller with increase of the diameter (width)-to-thickness ratio since the confining effect becomes smaller in this case. However, the strength index (SI) does not decrease significantly with increase of the diameter (width)-to-thickness ratio, which means that the diameter (width)-to-thickness ratio effect in the composite columns does not clearly appear, since the concrete core can prevent the local buckling. It was found that, from the viewpoint of the maximum strength, the current limitation of diameter (width)-to-thickness ratio for the composite columns in AIJ [2], i.e. the limit value of 1.5 times that of unfilled steel tube, is appropriate.

7. Conclusions

The present study is an attempt to study the behaviour of SCC-filled HSS stub columns. On the basis of the results of this study, the following conclusions can be drawn within the scope of these tests:

- (1) 50 stub column test results, with a wide range of diameter (width)-to-thickness ratio for HSS sections filled with SCC, were reported in this paper.
- (2) A simplified model is developed for calculating the section capacity and the axial load versus axial deformation relationships of the composite stub columns.
- (3) Comparisons are made with predicted section capacity using existing codes, such as ACI-1999, AIJ-1997, AISC-LRFD-1999, BS5400-1979 and EC4-1994. It seems that the conclusion regarding predictions using existing design codes made for NC-filled HSS stub columns remains the same for SCC-filled HSS stub columns.
- (4) The test results were compared to examine the limiting diameter (width)-to-thickness ratio for the composite columns. It was found that, from the viewpoint of the maximum strength, the current limitation of diameter (width)-to-thickness ratio in AIJ-1997, i.e. the limit value of 1.5 times that of unfilled steel tube, is appropriate.

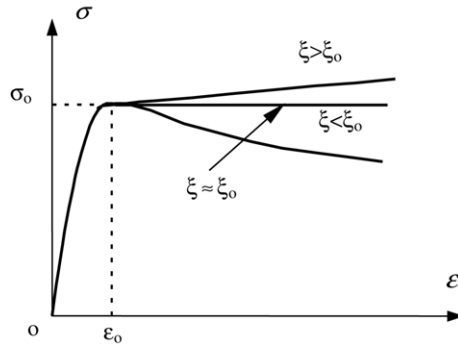


Fig. 21. A schematic view of axial stress (σ) versus axial strain (ε) relationship for concrete.

Acknowledgements

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Appendix A. Stress–strain relations of the confined concrete

A typical stress–strain curve for confined concrete is shown in Fig. 21, where the confinement factor (ξ) is defined as [12,10]

$$\xi = \frac{A_s \cdot f_{sy}}{A_c \cdot f_{ck}} = \alpha \cdot \frac{f_{sy}}{f_{ck}} \quad (\text{A.1})$$

in which A_s is the cross-section area of steel tube; A_c is the cross-section area of concrete; $\alpha = A_s/A_c$ is defined as the steel ratio; f_{sy} is the yield stress of steel tube, and f_{ck} is the compression strength of concrete. The value of f_{ck} is determined using 67% of the compression strength of cubic blocks. Detailed expressions are given in [10].

It can be seen from Fig. 21 that the higher the confinement factor (ξ), the higher the compression strength of confined concrete. It can also be seen from Fig. 21 that the higher ξ , the more ductile the confined concrete. The confinement factor (ξ), to some extent, represents the composite action of steel tubes and concrete core.

The stress (σ) versus strain (ε) relations for steel CHS confined concrete are given as

$$y = 2x - x^2 \quad (x \leq 1) \quad (\text{A.2a})$$

$$y = \begin{cases} 1 + q \cdot (x^{0.1\xi} - 1) & (\xi \geq 1.12) \\ \frac{x}{\beta \cdot (x - 1)^2 + x} & (\xi < 1.12) \end{cases} \quad (x > 1) \quad (\text{A.2b})$$

in which

$$x = \frac{\varepsilon}{\varepsilon_0};$$

$$y = \frac{\sigma}{\sigma_0};$$

$$\sigma_0 = \left[1 + (-0.054 \cdot \xi^2 + 0.4 \cdot \xi) \cdot \left(\frac{24}{f'_c} \right)^{0.45} \right] \cdot f'_c \quad (\text{A.3})$$

$$\varepsilon_0 = \varepsilon_{cc} + \left[1400 + 800 \cdot \left(\frac{f'_c}{24} - 1 \right) \right] \cdot \xi^{0.2} \text{ (}\mu\text{ε)};$$

$$\varepsilon_{cc} = 1300 + 12.5 \cdot f'_c \text{ (}\mu\text{ε)};$$

$$q = \frac{\xi^{0.745}}{2 + \xi};$$

$$\beta = (2.36 \times 10^{-5})^{[0.25 + (\xi - 0.5)^7]} \cdot f_c'^2 \cdot 3.51 \times 10^{-4}.$$

The units for stress and strain are N/mm² and $\mu\text{ε}$ respectively.

The stress (σ) versus strain (ε) relations for steel SHS confined concrete are given as

$$y = 2 \cdot x - x^2 \quad (x \leq 1) \quad (\text{A.4a})$$

$$y = \frac{x}{\beta \cdot (x - 1)^\eta + x} \quad (x > 1) \quad (\text{A.4b})$$

in which

$$x = \frac{\varepsilon}{\varepsilon_0};$$

$$y = \frac{\sigma}{\sigma_0};$$

$$\sigma_0 = \left[1 + (-0.0135 \cdot \xi^2 + 0.1 \cdot \xi) \cdot \left(\frac{24}{f'_c} \right)^{0.45} \right] \cdot f'_c \quad (\text{A.5})$$

$$\varepsilon_0 = \varepsilon_{cc} + \left[1330 + 760 \cdot \left(\frac{f'_c}{24} - 1 \right) \right] \cdot \xi^{0.2} \text{ (}\mu\text{ε)};$$

$$\varepsilon_{cc} = 1300 + 12.5 \cdot f'_c \text{ (}\mu\text{ε)};$$

$$\eta = 1.6 + 1.5/x;$$

$$\beta = \begin{cases} \frac{f_c'^{0.1}}{1.34\sqrt{1+\xi}} & (\xi \leq 3.0) \\ \frac{f_c'^{0.1}}{1.34\sqrt{1+\xi} \cdot (\xi - 2)^2} & (\xi > 3.0). \end{cases}$$

The units for stress and strain are N/mm² and $\mu\text{ε}$ respectively.

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