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Experimental study on concrete filled square hollow sections

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Abstract. A series of tests was performed to consider the behaviour of short composite columns under axial compressive loading, covering a range of S275 and S355 grade steel square hollow section filled with normal and high strength concrete. The interaction between the steel and the concrete component is considered and the results show that concrete shrinkage has an effect on the axial strength of the column. Comparisons between Eurocode 4, ACI-318 and the Australian Standards with the findings of this research were made. Result showed the equation used by the ACI-318 and the proposed Australian Standards gave better predication for the axial capacity of concrete filled SHS columns than the Eurocode 4.

Key words: composite column; concrete filled; square hollow sections; axial strength; confinement.

1. Introduction

Composite columns have been widely used in the construction industry for a number of years. Two main types of composite column can be used: concrete encased steel sections and concrete filled steel tubes including both square and circular cross-sectional tubes. This paper only deals with composite columns consisting of steel square hollow section (SHS) filled with concrete of various strengths.

This increase in use of the concrete filled steel columns throughout the world in recent years is mainly due to the significant advantages that this type of columns could offer in comparison to more traditional construction methods. Composite columns consist of a combination of concrete and steel, and make use of the best properties of these constituent materials. The use of composite columns can result in significant savings in column size, which ultimately can lead to significant economic savings. The reduction in column size can provide substantial benefits where floor space is at a premium such as in car parks and office blocks.

Composite columns are a very important application of composite construction. The principle of a column is to deliver vertical forces to the base of a structure, with the term 'composite column' referring to a compression member in which a steel element acts compositely with the concrete element. The role

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of the concrete core in a composite column is not only to resist compressive forces but also to reduce the potential for buckling of the steel member. The steel tube reinforces the concrete to resist any tensile forces, bending moments and shear forces. Composite columns can buckle by local or overall buckling, however this paper only considers short composite columns that had an overall length of 300 mm which meant that the columns were too short for overall buckling of the columns to occur and therefore only local buckling effects were exhibited.

The main aims have been to make a comparison between the tests and existing design codes including Eurocode (EC4), ACI-318 (ACI) and Australian Standards AS3600 & AS4100 (AS). This comparison includes considering the concrete cores with strengths of 30, 60 and 100 MPa and also considered the use of S275 and S355 grade SHS sections. Another area of consideration is the effect that shrinkage of the concrete core has on the ultimate strength of the composite column, this is an area where little research has been undertaken.

2. Past research

Composite encased columns have been used for over a hundred years, initially to provide fire protection to steel structures although later the strength properties were also taken into account. It was not until the 1960s that research into concrete filled steel tubes was first considered; a state of the art review of the major research undertaken into composite columns has been carried out recently by Shanmugan and Lakshmi (2001).

2.1. Axial strength

Composite columns respond to loading as a combination of the response of steel and concrete. This generally results in the composite interaction enhancing the performance of the whole to somewhat superior to the sum of the component parts. The axial strength that is achieved using composite columns is dependant on the column shape, size, confinement, strength of concrete and the bond generated between the steel and concrete.

Early research was undertaken by Gardner and Jacobson (1967) who investigated the behaviour of a total on twenty-two composite columns with *d/t* ratios varying between 30 and 40. Their results suggested that at ultimate load, the steel tube was at failure but the concrete core was not. They did note an increased strain level for the steel tube without local buckling, which suggested that the concrete stabilised the tube wall.

In the late 1960s, Neogi *et al.* (1969) developed a computer model to calculate the strength of slender columns. He successfully calibrated his model on his own test results using a number of specimens with *d/t* ratios between 40 and 80 under eccentric loading. Guiaux and Janss (1970) conducted a large number of tests on axially loaded thick-walled concrete filled steel tubes. The concrete used had unconfined cylinder strength of 42 MPa with the tubes been made of mild steel. Tests were performed on a series of column lengths including short columns. The aim of the research was to investigate the influence of column slenderness on specimen strength.

Lundberg (1993) assembled a selection of available experimental data for axially loaded columns. He compared the ultimate strengths with current design standards provided by AISC. The observed result was that the ultimate strength for the composite column exhibited a large variation; he concluded that further research was required.

Schneider (1998) undertook both an experimental and analytical study of the behaviour of short composite columns. In total fourteen specimens were tested investigating tube shape and wall thickness and considering how these affected the ultimate strength of a composite column. He suggested that circular tubes offer substantial post-yield strength and stiffness, although this effect is not available in most square or rectangular cross-sections.

Uy (2001) tried to determine how the axial strength of composite column varied with changing *b/t* ratios; the same steel tube thickness was used throughout which meant the column sizes were increased with the increases in the *b/t* ratio. It was determined that as the *b/t* ratio increases, so does the failure load due to the increase in concrete infill. One observation made during the test was that the lateral deformation of the test columns was insignificant in comparison to the applied eccentricity and hence they did not contribute to any secondary moments.

Eight tests were conducted by Wang (1999) on concrete filled 120×80×6.3 mm rectangular hollow steel slender columns. They were loaded with end eccentricities producing moments other than single curvature bending. A design method was also proposed, although details of his method were not given.

2.2. Concrete confinement

Saw and Liew (2000) stated that the reduction of concrete strength by 0.85 may be omitted for concrete filled composite columns in EC4 since the development of concrete strength is better achieved due to protection against the environment and against splitting of concrete. For composite concrete filled circular sections, the confinement effect of concrete increases the axial strength of the column as the concrete core is restrained laterally by the surrounding steel tube. Shanmugam and Lakshmi (2001) stated this increase in concrete strength outweighs the reduction in the yield strength of steel in vertical compression due to the confinement tension needed to contain the concrete.

Uy (1998) described the confinement effect in the following way. Concrete normally has a Poisson's ratio which is about 0.20 in the elastic range and increases as the concrete becomes non-linear and eventually crushes and softens. Steel has a Poisson's ratio of 0.30, which also increases to a value of 0.50 as plasticity develops and strain hardening occurs. If local buckling occurs prior to concrete crushing in a concrete-filled column, then the confinement effect is limited. However if local buckling is inelastic, then confinement of concrete can be developed.

It is generally accepted that there is little effect on the axial strength due to confinement of the concrete in square cross-sectional columns. However recent research by Hajjar and Gourley (1996) incorporated the effect of confinement into their model and compared this with experimental results with good accuracy for sections with small slenderness ratios. Kato (1996) performed thirteen independent tests on concrete filled steel rectangular columns. The study found that in determining the ultimate axial strength of the columns a 15% increase in concrete strength due to confinement occurred. This suggests that confinement also occurs in rectangular concrete filled steel columns. Each of these tests were carried out on columns with small plate slenderness ratios less than or equal to 40.

Knowles and Park (1969) studied a total of 21 circular and 7 square composite columns with *d/t* ratios of 15, 22 and 59 and *L/d* ratio's ranging between 2 to 21. It was noticed during the tests, for certain values of longitudinal strain, that the concrete component began to increase in volume as a result of micro cracking. This micro cracking induced concrete confinement by the steel tube. It was found that this confinement increased the overall load resisting capacity of the composite column with a circular cross-section. However, it was noticed that for columns with a large *L/d* ratio, the composite section failed by the columns buckling before reaching the strains necessary to cause an increase in concrete volume.

2.3. Bond stress between steel and concrete

In composite structures, the bond stress transfer between the steel and the concrete is critical to ensure composite action occurs. Most tests undertaken to look at bond stress capacities have been evaluated using push out test specimens. Research into the bond stress developed between the concrete and steel tube has been undertook by Roeder, Cameron and Brown (1999). They established that the bond stress demand varied depending upon the location within the structure. Demand was always greatest in regions of geometric discontinuity such as connections and foundation support, far less bond stress demand was required in connections where elements penetrate the concrete fill than in direct steel to steel connections. Roeder, Cameron and Brown (1999) state that the bond transfer between the steel tube and the concrete fill depends on the radial displacements due to the pressure of the wet concrete on the shell and the shrinkage of the concrete core, together with the rugosity (or internal surface irregularities) of the interior surface of the tube. The radial displacement due to the Poisson effect is significant only to the extent that composite action is not achieved, and the strains in the steel and concrete are different.

Shakir-Khalil and Mouli (1990) found from experimental research that the bond strength or bond stress for square section tubes varied between 0.39 to 0.51 N/mm^2 although this is relatively low when compared to those for reinforcement bars and circular hollow sections. The bond stress capacity is defined as the average interface stress associated with the initial rigid body slip of the concrete core relative to the steel tube. Virdi and Dowling (1973) showed that bond stress occurs due to the interlocking of the concrete and the steel, which is due to the surface roughness of the steel and the variation in shape of the tube cross section. Roeder, Cameron and Brown (1999) carried out a series of experiments to refine the equations for the average bond stress. They found there was a large reduction in bond stress capacity in larger diameter tubes and also that the bond stress decreased dramatically as the d/t ratio increases. They stated that the average bond stress for rectangular hollow sections was 70% smaller than for circular tubes and that the concrete compressive strength had no consistent effect on the bond stress capacity. The study showed that shrinkage can be very detrimental to the bond stress capacity and care must be exercised about the shrinkage potential of the concrete mix when the use of bond stress is being relied on in large diameter tubes. Evidence suggests that tubes with large d/t ratios and diameters lack the stiffness to enforce the benefits of irregularity in the cross section.

2.4. Use of high strength concrete

Uy (1998) stated that the use of high strength materials can provide many benefits to the construction of multi-storey buildings; one of these includes the reduction of the cross-section size and a subsequent increase in the lettable floor area. Gibbons and Scott (1996) showed that the effects of concrete confinement by using high strength concrete increases the column stiffness and hence increases in resistance to lateral loading in tall buildings.

Rangan and Joyce (1992) looked at the use of high strength concrete with thick wall mild steel tubes. The loading conditions were that the specimens were eccentrically loaded and were bent in symmetrical curvature, both column slenderness and load eccentricity were considered and an equation for calculating the column strength were proposed. Kilpatrick and Rangan (1997) performed 25 tests on composite columns using a high strength concrete. The columns that they used were circular in cross-section in order to study the influence of eccentricity of force upon the composite columns strength and load-deflection response and verified the proposed equation. All the specimens tested had d/t ratios of 42.

Prion and Boehme (1989) performed a series of tests with different loading condition including axial, eccentric and moment loading conditions. The tests were performed on steel sections with circular cross-sections with steel tube strengths of 250 and 330 N/mm^2. A series of composite columns were tested with varying lengths including ten short columns with high strength concrete cores in the order of 70 to 92 MPa. They found the failure mode of short axially loaded specimens was a diagonal shear failure across the specimen with reserve strength of approximately 60% of the ultimate strength.

2.5. Concrete shrinkage

Research into concrete shrinkage was performed by Lahlou (1997). He tested the effect of creating a physical boundary between the steel and the concrete so that no bonding occurred between the two materials and effectively simulating the effects of concrete shrinkage. The concrete core was cast using high strength concrete. The tests used circular cross-sectional steel tubes, mainly with steel thicknesses of 2 and 3 mm although thicker tube sections of 8 mm were used. Three basic tests were performed; one test was where both the steel and concrete were loaded. The other two tests simulated concrete shrinkage where one of the columns had a physical barrier between the steel and concrete to simulate shrinkage and the other did not. The columns that were tested to compare the effect of concrete shrinkage were loaded by the concrete core only. It can be seen concrete shrinkage has an effect on the ultimate strength of the column in the order of about 5% reduction in the columns' ultimate strength due to concrete shrinkage. This was performed on composite columns where only the concrete core was loaded.

Table 1 Specimen properties

Ref.	$D \times b \times t$ (mm)	Length, L (mm)	b/t	Concrete Cube Strength f_{cu} (MPa)	Steel Strength f_y (MPa)	Constraining factor ξ
S1	100×100×9.4	299	10	-	400	-
S2	100×100×4.8	299	20	-	289	-
S3	100.7×100.7×4.8	301	20	30.8	400	9.1
S4	101×101×9.6	300	10	93.6	400	3.0
S5	99.9×99.9×4.9	301	20	30.8	289	2.8
S6	99.8×99.8×4.9	300	20	93.6	289	0.9
S7	100.1×100.1×4.2	301	25	34.7	333	2.2
S8[a]	100×100×4.2	302	25	34.7	333	2.2
S9	100×100×4.1	299	25	97.2	333	0.8
S10[a]	100×100×4.1	300	25	97.2	333	0.8
S11	100×100×4.1	300	25	-	333	-
S12	100×100×4.1	301	25	57.6	333	1.3
S13[a]	99.9×99.9×4.0	301	25	57.6	333	1.3
S14	101×101×9.6	302	10	57.6	400	4.9
S15[a]	99.8×99.8×4.8	302	20	31.9	289	2.7
S16	99.7×99.7×4.7	301	20	58.2	289	1.5
S17[a]	99.7×99.7×4.73	302	20	98.9	289	0.9
S18	99.9×99.9×4.1	301	25	98.9	333	0.8

[a]Indicates *Greased* columns.

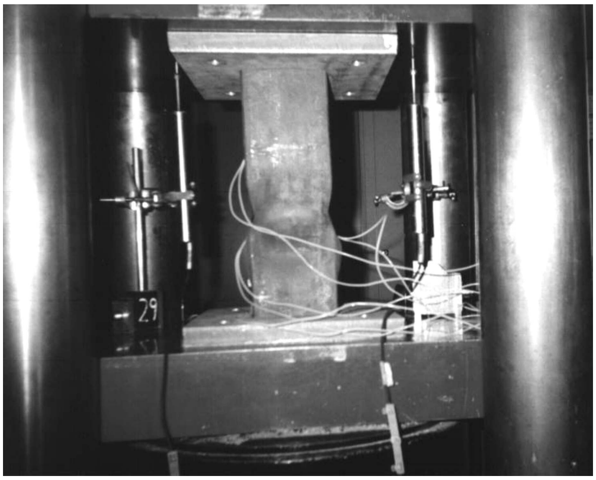

Fig. 1 Test arrangement and instrumentations

3. Experimental programme

In order to consider the behaviour of the composite concrete filled column, 18 specimens were tested with various concrete strength and wall thickness. 100×100 mm hot-finished square hollow section (SHS) with 4.0 mm, 5.0 mm and 10.0 mm wall thickness were used for the tests. All specimens were 300 mm in length to reduce end effects and to ensure that the specimens would be stub columns with little effect from column slenderness. The properties for all the specimens are listed in Table 1. Testing of the columns was carried out using a 3000 kN capacity Toni Pack 3000 testing machine. The experimental set up is shown in Fig. 1. Both ends of the specimens were milled flat and capped with rigid steel caps to distribute the applied load uniformly over the concrete and the steel section.

3.1. Concrete properties

A 30, 60, and 100 N/mm^2 concrete were produced using only commercially available materials with normal mixing and curing techniques. Three trial mix designs had been prepared before the start of the experiments. The mix designs are shown in Table 2. Standard cube tests were used to determine the compressive strength for the concrete in accordance to the British Standards. The cube strength and the development curves are shown in Table 3 and Fig. 2 respectively.

3.2. Steel properties

In order to determine the strength of the steel tubes, tensile coupon tests were carried out for each wall thickness. From the tests, the yield strength, f_y of the steel sections can be obtained. The strength of the steel section is given in Table 1. A typical stress-strain curve for the 4 mm section is shown in Fig. 3.

Table 2 Concrete mix design

Cast	Proposed strength (MPa)	W/C ratio	Mix proportions					
			Cement	Sand	Gravel	Water	Silica fume	Super plasticiser
Trial	30	0.65	1.0	2.5	3.5	0.65	0	0
Trial	60	0.40	1.0	2.0	3.25	0.40	0	0
Trial	100	0.28	0.9	1.5	2.5	0.28	0.1	3%
1	30	0.65	1.0	2.5	3.5	0.65	0	0
1	100	0.28	0.9	1.5	2.5	0.28	0.1	1%
2	30	0.65	1.0	3.0	3.5	0.65	0	0
2	100	0.28	0.9	1.5	2.5	0.28	0.1	2%
3	60	0.42	1.0	2.0	3.25	0.42	0	0
4	30	0.65	1.0	3.0	3.50	0.65	0	0
4	60	0.40	1.0	2.0	3.25	0.40	0	0
4	100	0.28	0.9	1.5	2.50	0.28	0.1	2%

Super plasticiser = %weight of cement

Table 3 Concrete cube strength

Cast	Target strength MPa	Strength MPa				
		7 days	8 days	14 days	20 days	28 days
1	30	30.8	-	38.9	-	46.2
2	30	34.7	-	39.6	-	45.9
4	30	31.9	-	-	-	45.0
3	60	-	57.6	59.1	-	64.7
4	60	58.2	-	-	-	67.4
1	100	-	-	93.6	101.5	104.7
2	100	-	-	97.2	104.9	105.8
4	100	-	-	98.9	-	108.6

3.3. Testing procedure

The specimens loaded at 50 kN intervals at the beginning of the test (i.e., in the elastic region) and at a loading rate of 10 kN intervals after the column began to yield, in order to have sufficient data points to delineate the “knee” of the stress-strain curve. All the operation and the change of loading rate were operated manually. All the readings were recorded when both load and strain had been stabilised. After the immediate drop of the load due to local duckling, the test continued as the load stabilised until the load started again to increase slightly when the testing ended. Then the specimen was removed and carefully examined after the test.

3.4. Instrumentation

Two linear variable differential transducers (LVDTs) were placed in diametrically opposite position

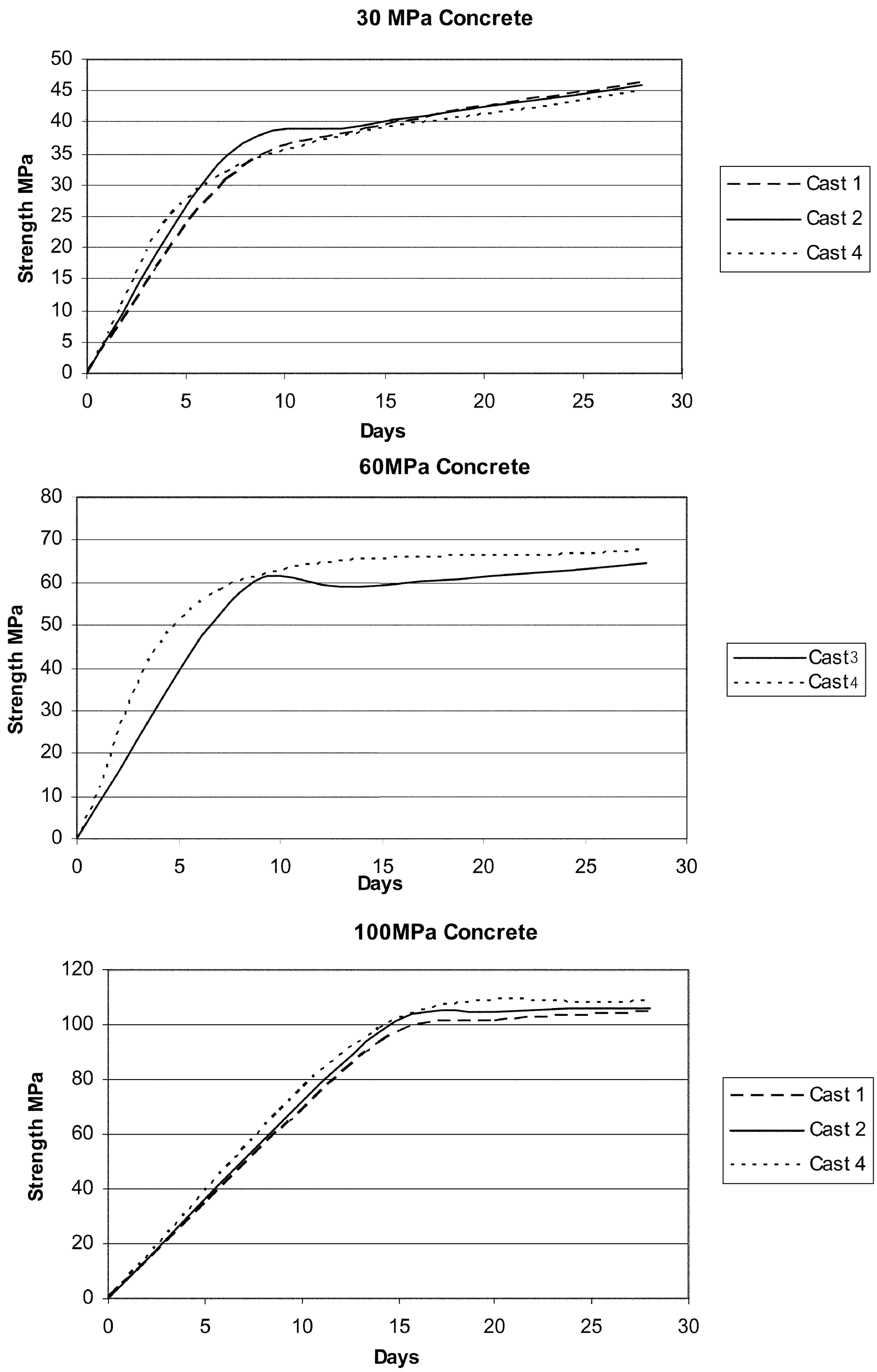

Fig. 2 Concrete strength development curves

equally spaced at each side of the column to monitor the overall deformation (Fig. 1). Axial deformations were obtained from the average of the LVDTs for any specimen.

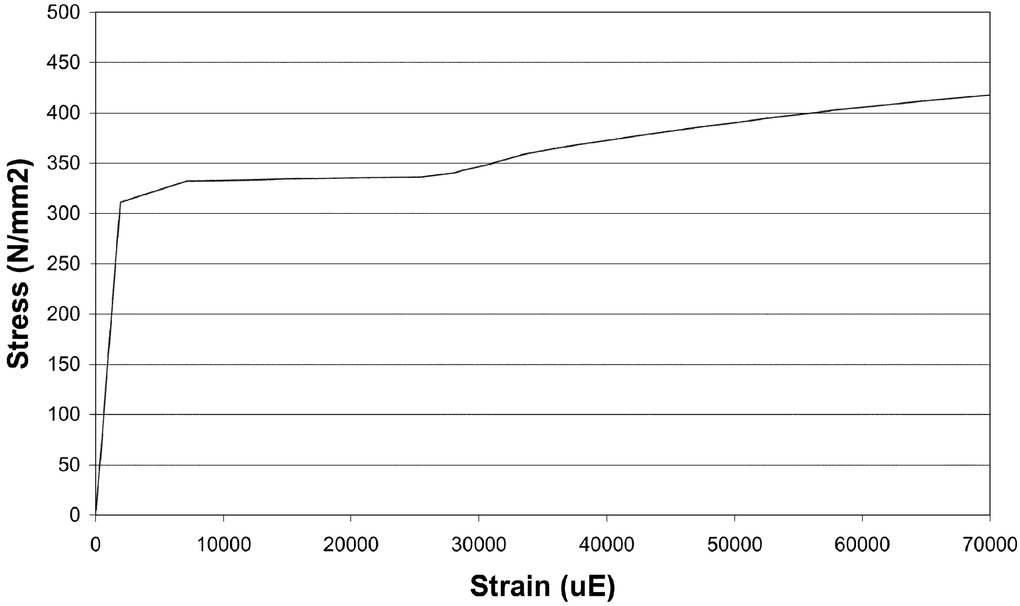

Fig. 3 Stress vs strain curve of 4 mm SHS

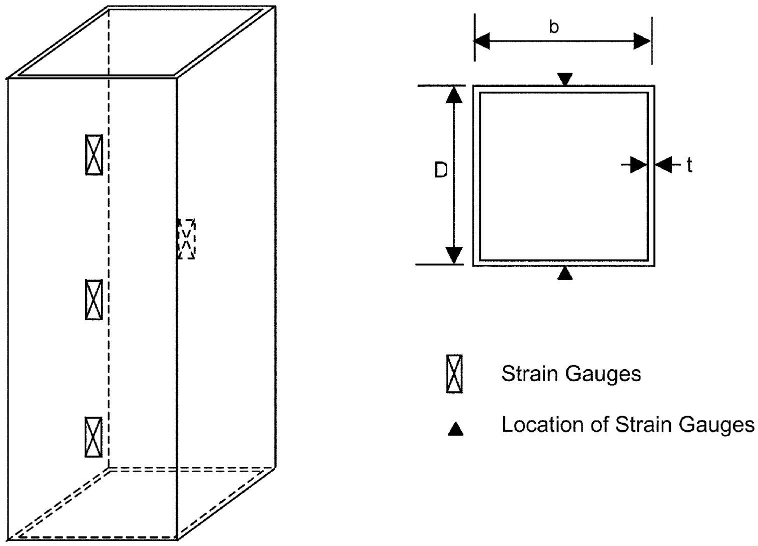

Fig. 4 Strain gauges location

Three strain gauges were placed on one side and one strain gauge was placed on the diametrically opposite side as shown in Fig. 4. All of the strain gauges were placed on the exterior of the concrete filled tubes. On side A, the top and the bottom gauges were at distance of 5 0mm from the top and the bottom of the column respectively. The middle strain gauges on sides A and B were positioned at the mid-height of the column. All the data from the LVDT's, Loadcell and strain gauges were recorded and stored in the data logger.

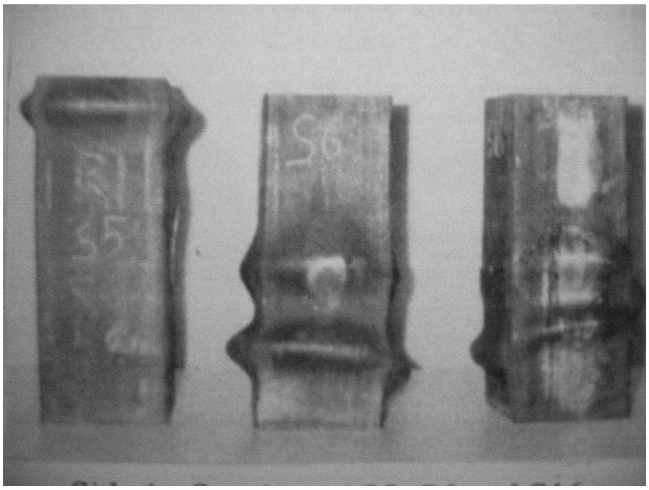

Fig. 5 Typical mode of failure

4. Test results

Typical modes of failure of the composite concrete-filled SHS column under axial compression are shown in Fig. 5. Load versus axial displacement of all the tests are shown in Fig. 6. The effects of bonding between steel and concrete to the axial capacity of the composite column causes by concrete shrinkage were considered by making a comparison between Greased and Non-Greased sections.

In comparing the load-displacement curves for Non-Greased and Greased specimens for each concrete strength (Figs. 7, 8 & 9), it is shown that for 30 MPa concrete filled strength, the two curves for Greased and Non-Greased specimen are more or less the same. The axial capacity of both specimens was approximately 700 kN for the 4 mm thick tube and 800 kN for the 5 mm thick tube. The variation on axial capacity for Non-Greased and Greased specimens is slightly larger for 60 MPa concrete (Fig. 8), a reduction of 5.7% in the ultimate column strength of Greased specimen to the Non-Greased counterpart was recorded. The bonding of the steel and concrete would seem to affect the 100 MPa high strength concrete most, the percentage of difference between the Non-Greased and Greased specimens is 14%. The effect of bond between the concrete and the steel tube on the axial capacity of the composite column with 100 MPa is shown in Fig. 9.

By comparing Figs. 7, 8 and 9, it is concluded that the effects of the bond between the concrete and the steel tube is more critical for high strength concrete. For normal strength concrete, the reduction on the axial capacity due to the loss of bonding between steel and concrete is negligible.

A constraining factor (ξ), to some extent, represents the ductility and composite action between the steel SHS and concrete is introduced. The constraining factor (ξ) is defined as:

$$\xi = \frac{A_s f_y}{A_c f_c}$$

where

A_s is the cross-section area of the steel section,

A_c is the cross-section area of concrete,

f_y is the yield strength of the steel section,

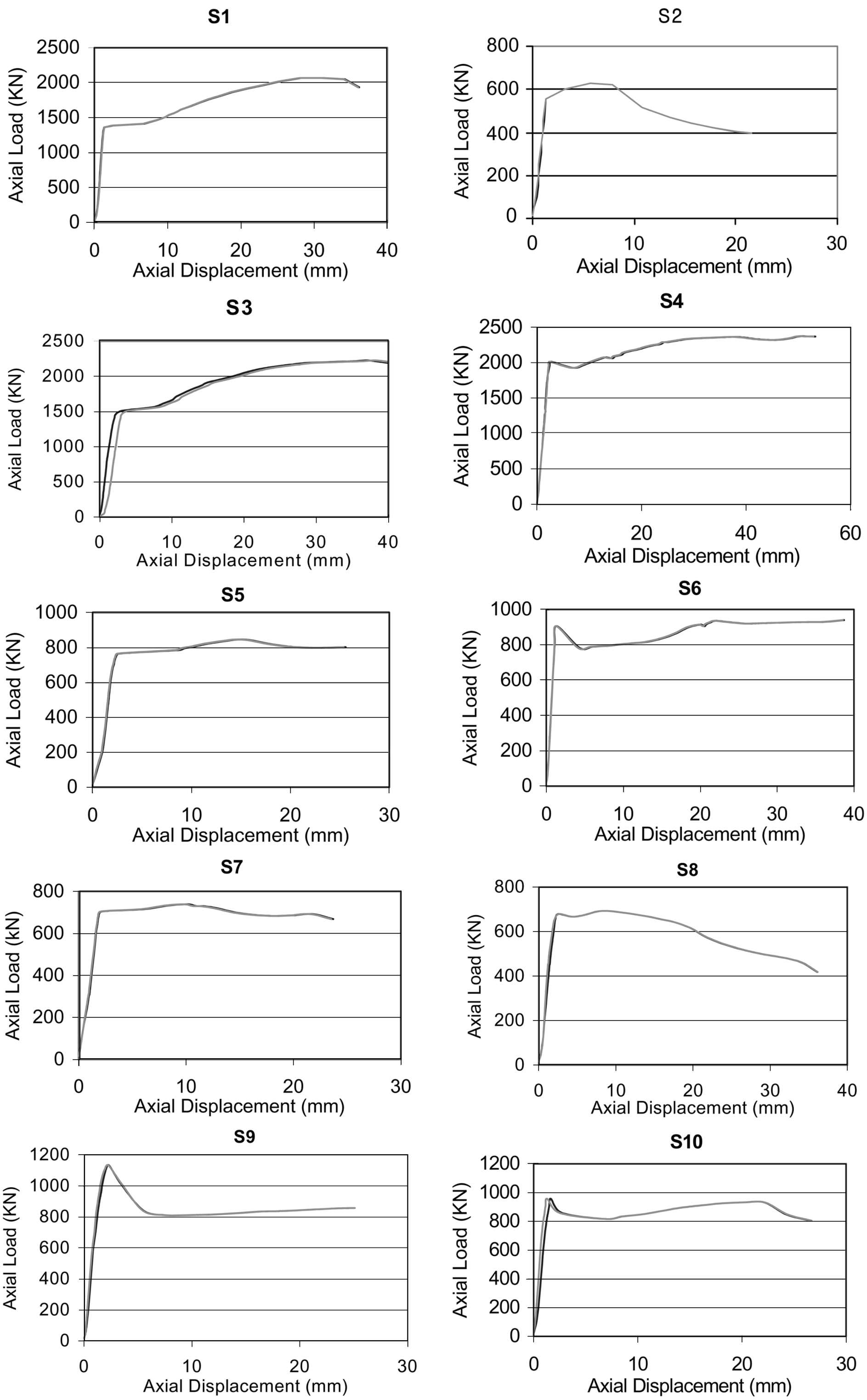

Fig. 6 Axial load versus axial displacement

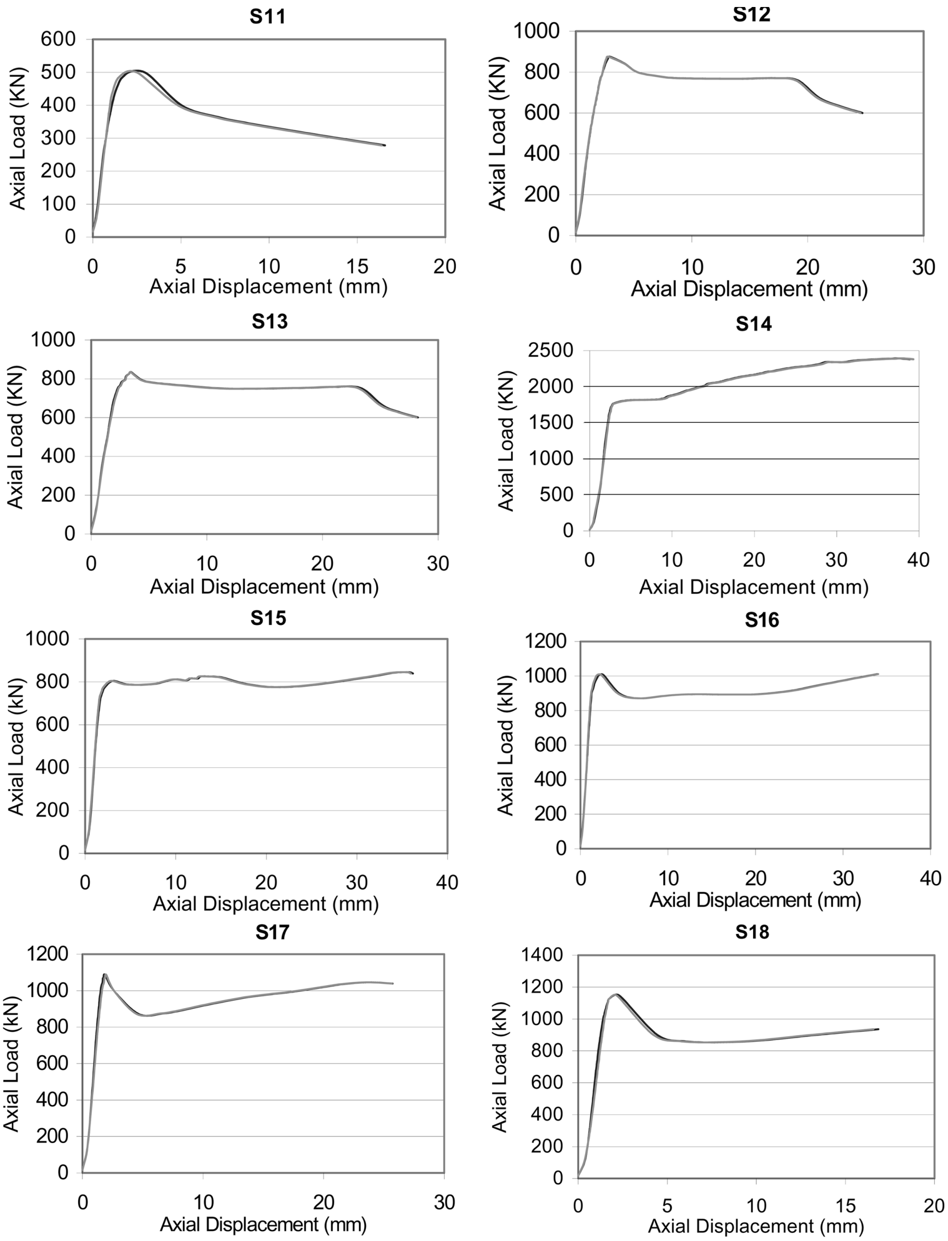

Fig. 6 Continued

f_c is 0.8 × concrete cube strength.

The constraining factor for all the test specimens are shown in Table 1. From Fig. 6, it can be observed that the curves tend to drop quickly after the ultimate load is reached for specimens with constraining factor lower than 2.0, whereas the load tends to maintain after the ultimate load for specimens with constraining factor larger than 2.0. This would suggest the contribution from the steel section has a direct effect to the ductility of the composite SHS columns.

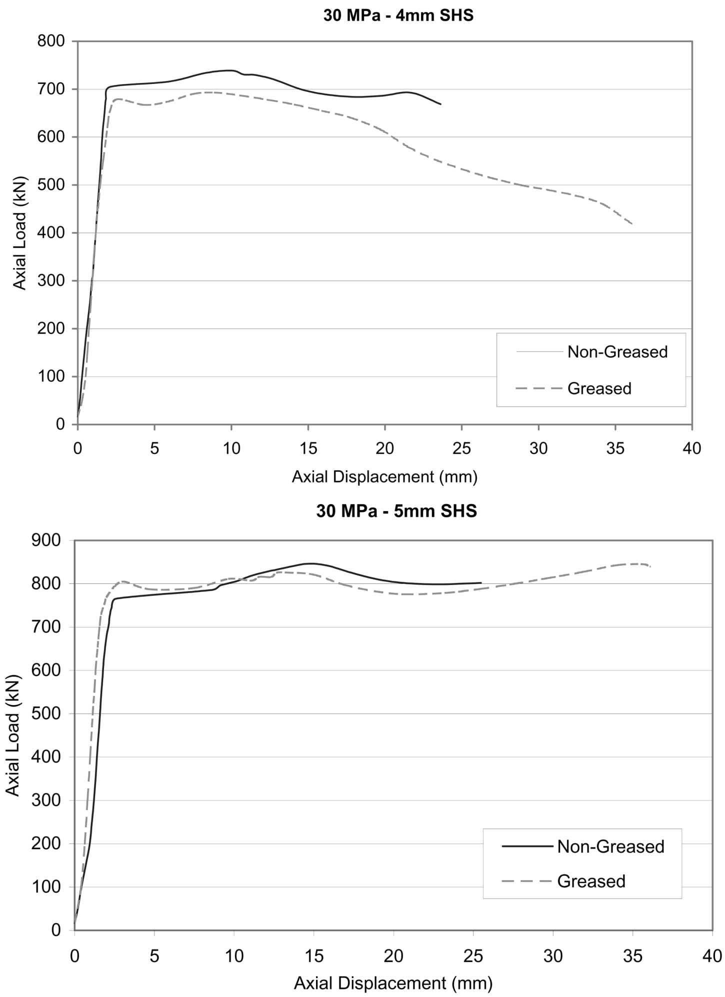

Fig. 7 Axial load-displacement, Non-Greased and Greased, 30 MPa concrete

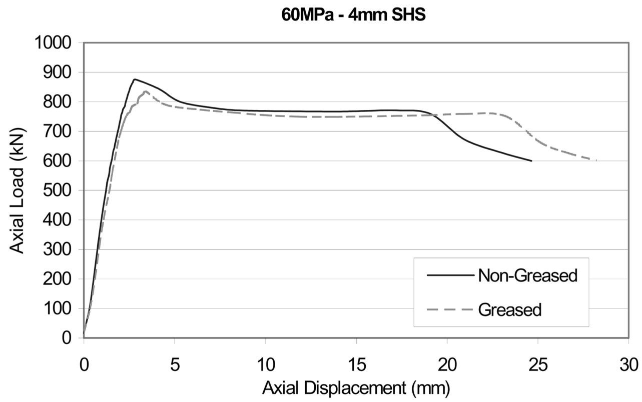

Fig. 8 Axial load-displacement, Non-Greased and Greased, 60 MPa concrete

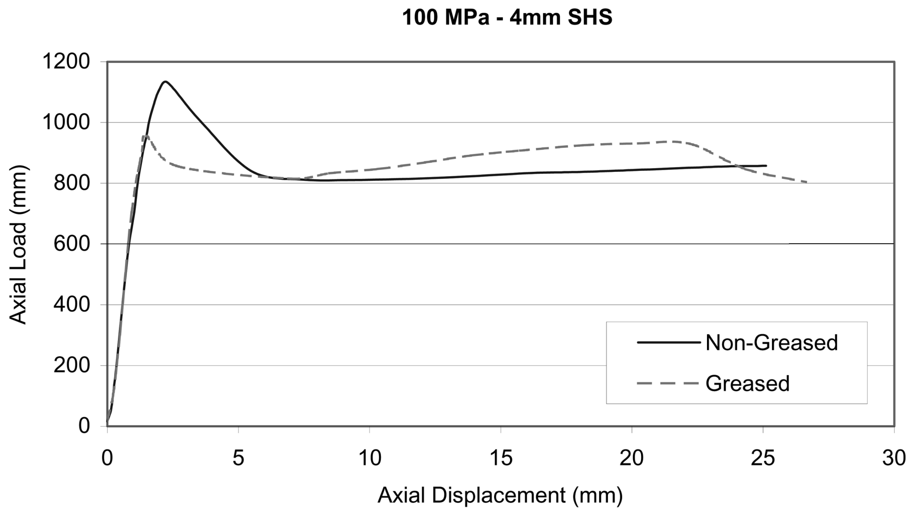

Fig. 9 Axial load-displacement, Non-Greased and Greased, 100 MPa concrete

5. Comparison with Eurocode 4 (EC4), ACI 318-95 (ACI) and Australian Standards AS3600 & AS4100 (AS)

EC4 is the most recently international standard in composite construction. EC4 covers concrete encased and partially encased steel sections and concrete filled sections with or without reinforcement. EC4 uses limit state concepts to achieve the aims of serviceability and safety by applying partial safety factor to load and material properties. The ultimate axial force of composite concrete-filled column is:

$$N_{plRd} = A_c\ f_c + A_s\, f_y \tag{1}$$

The ACI and Australian Standards use the same formula for calculating the squash load. Both codes do not take into consideration the concrete confinement. The limiting thickness of steel tube to prevent local buckling are based on achieving yield stress in a hollow steel tube under monotonic axial loading which is not necessary requirement for in-filled composite column.

The squash load is determined by:

$$N_u = 0.85\, A_c\, f_c\ + A_s\, f_y \tag{2}$$

The ultimate axial loads of all the tests are compared with the predicted load from EC4, ACI and AS are shown in Table 4. EC4 over-estimated the experimental values for some of the Greased column by up to 20%, which suggested the reduction due to the absence of the bond between the concrete and the steel tube, are significant especially with SHS filled with high strength concrete. Figs. 10 and 11 show the comparison of EC4 with experimental results, for both Non-Greased and Greased specimens. From the results, there is little evident to suggest that confinement of the concrete by the steel tubes are presented.

Table 4. Comparison of test results

Ref.	A_s (mm^2)	A_c (mm^2)	Steel strength, f_y (MPa)	Concrete strength (MPa)		N_{test} (kN)	N_{EC4} (kN)	$N_{ACI/AS}$ (kN)	N_{test} / N_{EC4} (kN)	N_{test} / $N_{ACI/AS}$ (kN)
				f_{cu}	f_c					
S1	3600	0.0	400	-	-	1400	1440	1440	0.97	0.97
S2	1900	0.0	289	-	-	550	549	549	1.00	1.00
S3	3600	6400	400	30.8	24.6	1550	1598	1574	0.97	0.98
S4	3600	6400	400	93.6	74.9	2000	1919	1847	1.04	1.08
S5	1900	8100	289	30.8	24.6	800	749	719	1.07	1.11
S6	1900	8100	289	93.6	74.9	900	1156	1065	0.78	0.85
S7	1536	8464	333	34.7	27.8	700	746	711	0.94	0.98
S8[a]	1536	8464	333	34.7	27.8	680	746	711	0.91	0.96
S9	1536	8464	333	97.2	77.8	1130	1170	1071	0.97	1.06
S10[a]	1536	8464	333	97.2	77.8	970	1170	1071	0.83	0.91
S11	1536	0.0	333	-	-	500	511	511	0.98	0.98
S12	1536	8464	333	57.6	46.1	880	902	843	0.98	1.04
S13[a]	1536	8464	333	57.6	46.1	830	902	843	0.92	0.98
S14	3600	6400	400	57.6	46.1	1800	1735	1691	1.04	1.06
S15[a]	1900	8100	289	31.9	25.5	780	756	725	1.03	1.08
S16	1900	8100	289	58.2	46.6	1000	926	870	1.08	1.15
S17[a]	1900	8100	289	98.9	79.1	1050	1190	1094	0.88	0.96
S18	1536	8464	333	98.9	79.1	1130	1181	1081	0.96	1.05

[a]Indicates *Greased* columns.

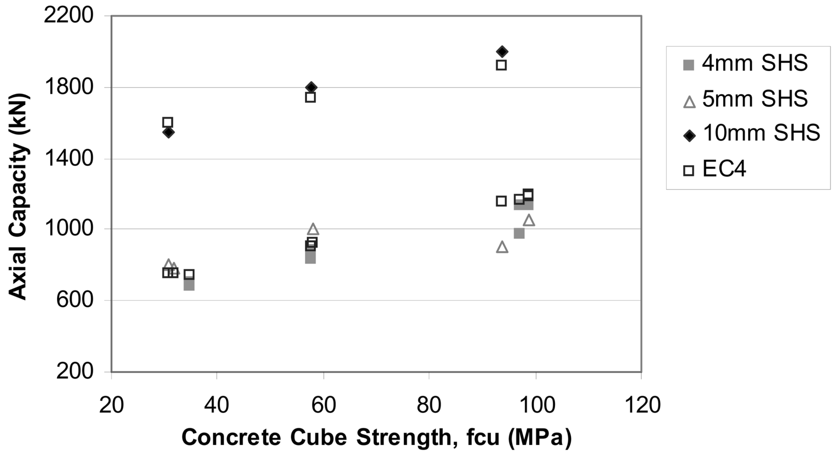

Fig. 10 Comparison of Eurocode 4 with experimental results

All the experimental results for Non-Greased composite columns (except S6 & S7) are higher than that calculated from the ACI / AS equation. Figs. 12 and 13 show the variation between the values from tests and that from the ACI / AS codes are constant in all the range of the concrete strength. The average N_{test} / $N_{ACI/AS}$ for *Greased* column is 0.98 when for *Non-Greased* is 1.04.

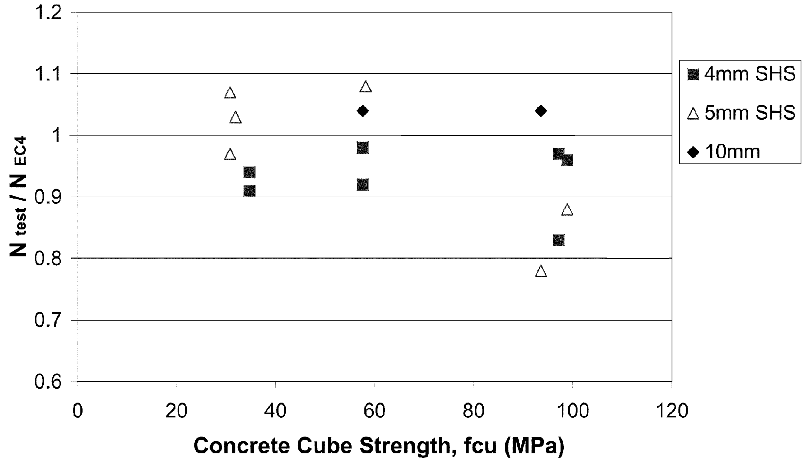

Fig. 11 N_{test} / N_{EC4} vs concrete cube strength

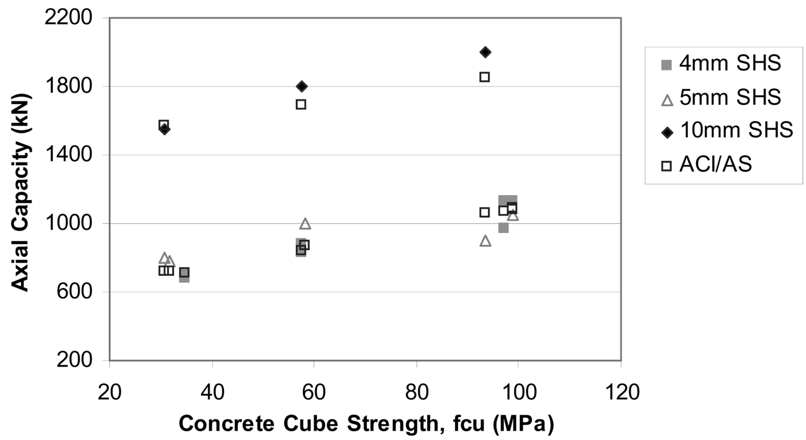

Fig. 12 Comparison of ACI and AS with experimental results

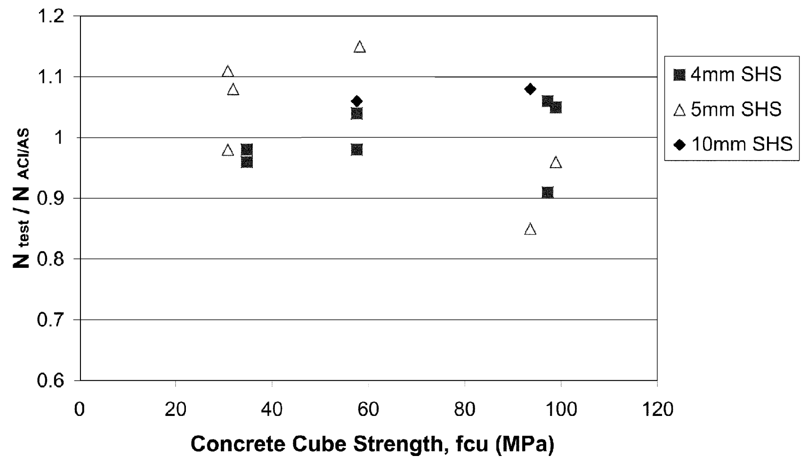

Fig. 13 $N_{test} / N_{ACI/AS}$ vs concrete cube strength

5. Conclusions

This paper presents the findings of 18 short square concrete filled steel tube columns tested under axial load. The results show that peak load was achieved at small shortening (≈ 3.0 mm) for composite columns with low constraining factor, where as for composite columns with high constraining factor, the ultimate load was maintained with large displacement.

As the concrete strength increases the effects of the bond of the concrete and the steel tube became more critical. For normal concrete strength, the reduction on the axial capacity of the column due to bonding was negligible. For high strength concrete, the variation between *Non-Greased* and *Greased* was 14%.

Eurocode 4 over-estimated the axial strength of concrete filled SHS columns, 20% was the largest difference between the experimental and calculated value on the axial capacity. The predicted axial strengths using ACI-318 and Australian Standards were general lower than the results obtained from experiments. Excellent prediction was achieved for concrete filled SHS columns with an average N_{test} / $N_{ACI/AS}$ ratio around unity.

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Notation

A_c : cross-sectional area of concrete
A_s : cross-sectional area of steel tube
b, B : external width of the steel tube
d, D : external depth or diameter of steel tube
f_c : compressive strength of concrete
f_{cu} : concrete cube strength
f_y : yield strength of steel tube
L : length of column
N_u : ultimate squash load
N_{plRd} : ultimate axial strength of composite column
t : wall thickness of steel section
SC