COLUMN BUCKLING TESTS OF HOT-ROLLED CONCRETE FILLED SQUARE HOLLOW SECTIONS OF MILD TO HIGH STRENGTH STEEL

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Abstract:

This paper examines the behaviour of 29 square concrete-filled steel tubes (CFST) under concentric axial compression. The cross-section of the hot-rolled steel tubes ranges from 60x60x3.0mm to 150x150x4.5mm, and are grouped in three series (Series 1, 2 and 3). For the sections tested, the slenderness ratio (L/b) for Series 1, 2 and 3 ranges from 6.67 to 24.61, 27.00 to 45.00 and 18.00 to 27.0, respectively; and the depth-to-wall thickness (b/t) ranges from 33.87 to 42.86, 13.33 to 33.33 and 22.22 to 50.00, respectively. The slenderness ratio provides a range of columns, from relatively short to slender composite columns. Particular attention is paid to the existence of circumferential strain in the square composite columns. The compressive resistance of the composite columns are compared with the design strengths predicted by the South African standard (SANS 10162-1), as well as the European design standard (EN 1994-1-1). SANS10162-1 is based on the Canadian steel code (CAN/CSA-S160-01). A two-stage equation is proposed to model the results of short columns (Equation 4) and intermediate to slender columns (Equation 5).

Keywords: Square concrete-filled steel tubes, hot-rolled, slenderness ratio, depth-to-wall thickness, overall and local buckling, confinement, circumferential strain.

1. Introduction

Slender rectangular composite columns have been the subject of much research over the years. These studies include hot-rolled concrete-filled rectangular hollow steel sections (RHSSs) by Shakir-Khalil and Zeghiche [1], Shakir-Khalil and Mouli [2], and Wang and Moore [3] and fabricated concrete-filled RHSS by Matsui et al [4], Han and Yao [5,6], Vrcelj and Uy [7,8] and Mursi and Uy [9,10].

Shakir-Khalil and Zeghiche [1] tested 7, 2.76m long hot-rolled RHSS composite columns of 120x80x5 cross-section. One column was subjected to axial loading, two to uniaxial bending about the major axis, two to uniaxial bending about the minor axis, and two to biaxial bending. To create the moment in the column, load eccentricity ranged from 0.0 to 60mm. The strength of the steel varied from 343MPa to 386MPa and the strength of the concrete was 42.85MPa. Based on these tests it was concluded at the time that BS 5400 [11] gave safe predictions for axially loaded column as well as for columns in uniaxial bending about the major axis. However, in the case of uniaxial bending about the minor axis, both theory and tests showed that the BS 5400 [11] predictions were on the unsafe side. The failure mode of all columns was an overall buckling mode, with no sign of local buckling of the wall of the steel section.

Further tests on 9, 3m long hot-rolled concrete-filled RHSSs of 120x80x5mm (5 tests) and 150x100x5mm (4 tests) cross-section were performed by Shakir-Khalil and Mouli [2], to illustrate the increase in the strength of bare steel rectangular hollow sections when they are filled with concrete. Short composite columns were also tested in order to establish the squash load of stub columns. The average cube strength of concrete ranged from 40.8MPa to 47.2MPa and the average yield strength of the steel varied from 340MPa to 363MPa for the nine specimens. The load eccentricities of the 120x80x5mm and 150x100x5mm concrete-filled RHSSs about both axes were (0;8), (12;8), (42;28), (24;40), (60;16), and (0;0), (15;10), (45;30) and (75;50), respectively. Comparison of the tests strengths with BS5400 [11] predictions showed that filling the section with concrete increased the capacity of the 120x80x5mm columns by 12% and 150x100x5mm columns by 65%. Failure of columns took place by overall flexural buckling of the column with no sign of local buckling. With the exception of uniaxial bending about the minor axis for which the BS 5400 [11] predictions were found to be unconservative, the experimental results were in general agreement with BS 5400 [11].

Wang and Moore [3] performed 8 tests on 4m long, hot-rolled concrete-filled RHSS columns of 120x80x6.3mm cross-section, with two columns bent about the minor axis, two columns bent about major axis, and four columns subjected to biaxial bending. The average yield stress of the RHSS was found to be 370MPa and the average cube strength 60MPa. Except for Test 8, which buckled suddenly about the minor axis, all other columns failed in a gradual manner by elastic-plastic buckling about the minor axis. These tests, together with a comprehensive range of published experimental data on composite columns, were used develop a simplified method to design these columns. The method was based on the recommendations of BS 5950-1 [12] for steel columns and its column strengths predictions were in reasonable agreement with test results. In addition, the strengths predicted from this procedure were comparable with the predictions BS 5400 [11] and BS EN 1994-1-1 (EC4) [13].

Matsui et al. [4] tested six rectangular concrete filled steel tube (CFST) columns as part of a test series that comprised both columns and beam-columns. The steel portion was mild steel of 411.62MPa yield strength and the depth-to-thickness ratio of the steel plate element was 35.1. In all columns the average compressive strength of concrete was 40.69MPa and the effective length of columns ranged from 600mm to 4956mm. A more accurate method, with larger concrete column strength, was proposed for the Architectural Institute of Japan (AIJ, [14]) by the authors. The difference between the old and new AIJ method was only in the strength of the concrete column.

Han and Yao [5] tested 23, 133x133x4.5mm concrete-filled square hollow steel section (SHSS) columns (4 concentrically loaded and 19 eccentrically loaded) to determine the effects of pre-load on the steel tubes during construction. Three tension coupons, cut from the steel sheet produced an average yield strength of 340MPa, and the average concrete strength at 28 days was 42.2MPa. The tubes were manufactured from 4 steel plates, cut from the 4.5mm mild steel sheet and tack welded into a square shape and then welded with a single bevel butt weld at the corners. In these tests, the pre-load ratio ranged from 0 to 0.7, load eccentricity from 0 to 31mm and column slenderness from10 to 40 (length ranged from 465–2793mm). In general, the pre-load on the steel tubes resulted in an increase in column deformation, reduced column stiffness and ultimate strength of long columns. Design expressions for evaluating the ultimate strength of concrete-filled steel tubular columns with the steel tubes subjected to pre-load were developed.

Han and Yao [6] studied the influence of concrete compaction on the strength of 35, 130x 130x2.65mm to 360x240x2.65mm concrete-filled SHSS columns (23 concentrically loaded and 12 eccentrically loaded). Variables in the tests included the depth-to-width ratio of the column (1.0-2.0), tube depth to thickness ratio (34-136), load eccentricity (0 to 31mm) and column slenderness (21-62, corresponding to a length of 540–2340mm). All steel tubes were manufactured from 2.65mm mild steel sheet, with four plates cut from the sheet, tack welded into a rectangular shape and then welded with a single bevel butt weld at the corners. Three coupons, cut from each face of the steel tube produced an average yield strength of 340.1MPa, and the concrete strength at 28 days was 23.1MPa. It was found that better compaction of concrete resulted in higher member capacities in concrete-filled RHSS columns.

Vrcelj and Uy [7,8] and Mursi and Uy [9,10] performed comprehensive experimental and numerical studies to determine the effect of a slender cross-section on the overall buckling capacity (coupled local and global buckling) of slender thin-walled concrete filled steel box columns over a wide range of sections. In the first investigation Vrcelj and Uy [7, 8] tested 2, 65x65x3mm and 2, 75x75x3mm composite square sections, and all were 1.77m long. Each pair of section was tested with two different strengths of steel (400 and 450MPa yield strength) and concrete (52MPa and 79MPa). To complement these few tests, data from previous investigations (Bridge [15], Shakir-Khalil and Zeghiche [1], Shakir-Khalil and Mouli [2], and Wang and Moore [3]) were included in the numerical analysis. The second investigation (Mursi and Uy [9]) consisted of 2.8m long, 104x104x3mm, 134x134x3mm and 164x164x3mm slender thin-walled concrete filled steel box columns, and the average strength of concrete and steel was 65MPa and 269MPa (yield), respectively.

In the third investigation (experimentally and numerically) Mursi and Uy [10] tested four (4) composite columns, 110x110x5mm, 160x160x5mm, 210x210x5mm and 260x260x5mm, at a loading eccentricity of 10mm, 15mm, 20mm and 25mm, respectively. All columns were 3.02m long. Tests on coupons and concrete (at 28 days) in this study produced an average strength of 761MPa and 20.34MPa, respectively. All sections in these tests were fabricated by welding steel plates together using longitudinal fillet welds along the full length of the columns. Based on this extensive study a unified model was presented to address interaction buckling modes considering the coupled effects of confinement and local buckling in the elastic and inelastic range for concrete filled steel columns. The results were compared to EC4 [13], and design recommendations were proposed.

Despite the numerous publications of slender concrete-filled RHSSs available in literature, most of the investigations were either performed on thicker hot-rolled steel sections [1-3], which experiences less local buckling than the thin hot-rolled hollow sections considered in this paper or on fabricated steel sections [4-10], which have non-uniform cross-sections, high residual stresses and potential regions of weakness because of the welding process. Thin steel tubes can reduce the local stability of the steel section once overall flexural takes place. In addition it seems as if the design formulae used in square thin hot-rolled CFST is based on expressions developed from thick hot-rolled or fabricated RHSSs. A comparison of the experimental results with the loads predicted by the South African code (SANS10162-1) and Eurocode 4 (EC4) of circular concrete filled steel tube columns in compression by the author showed that the codes are conservative [16]. This paper provides tests on square slender hot-rolled columns with slender steel cross-sections in order to develop design expressions for such columns.

2. Material properties

2.1 Properties of steel sections

Steel properties were established by carrying out tensile tests on steel coupons, cut from hot-rolled square hollow sections under consideration. The coupons were taken from the flat portions of the sections only, and no beneficial effect in strength, caused by the strengthening of the corner regions of the cross-sections was considered. Coupon tests from the flat portions represent material properties of virgin steel sheet. All coupons were machined so that they had an overall length of 200 mm, overall width of 25 mm, and reduced sections of 100 mm long by 16 mm wide, as provided by the guidelines in the British Standard, BS EN ISO 6892-1 [17]. The coupons were tested at a rate of 3 mm/min according the guidelines of the same standard. An Instron 1195 machine with a capacity of 100kN was used for tensile coupon tests. An extensometer was attached to each coupon during the tests, to determine the strain.

The average steel's yield strength $\left(f_y\right)$, ultimate strength $\left(f_u\right)$ and elastic modulus $\left(E_s\right)$ of Series 1, 2 and 3 SHSSs are given in Table 1. These results were established from the stress-strain curves. Since the yield strength of the SHSSs ranges from 355MPa to 522.11MPa, the steel can be regarded as mild-to-high strength steel (SANS 10162-1 [18] and EN 1993-1-1 (EC3) [19]). The yield strength and elastic modulus obtained from the tests were used to predict the column strength using the South African code SANS 10162-1 [18] and EC4 [13]. SANS10162-1 [18] is based on the Canadian structural steel code (CAN/CSA-S160-09 [20])

Table 1: Average tensile coupon test results

Series	SHSS size (mm)	f_y (MPa)	f_u MPa)	f_u/f_y	ε_y	ε_u	$\varepsilon_u/\varepsilon_y$ (%)	E_s (GPa)
Series 1	101.6x101.6x3.0	483.90	552.70	1.14	0.0041	0.0951	23.20	204.7
	120.0x120.0x3.5	401.00	456.50	1.14	0.0045	0.1386	30.80	205.8
	150.0x150.0x3.5	405.00	481.10	1.19	0.0040	0.1569	39.23	203.7
Series 2	60.0x60.0x3.0	351.51	373.81	1.06	0.1758	17.1992	97.83	202.23
	60.0x60.0x4.0	528.55	538.83	1.02	0.2568	1.5610	6.09	207.40
	60.0x60.0x4.5	522.11	547.72	1.05	0.2561	6.4410	25.16	205.52
	63.0x63.0x3.0	442.69	479.38	1.08	0.2206	16.2817	73.84	202.10
	63.0x63.0x4.0	517.49	545.31	1.05	0.2546	11.5125	45.23	204.89
	63.0x63.0x4.5	449.95	480.05	1.07	0.2243	13.9583	62.25	210.40
	76.0x76.0x3.0	441.12	497.86	1.13	0.2192	14.2861	65.18	203.10
	76.0x76.0x4.0	498.38	528.64	1.06	0.2425	6.0685	24.98	206.80
	76.0x76.0x4.5	500.97	532.65	1.06	0.2420	9.3932	38.85	207.00
	100.0x100.0x3.0	441.19	493.92	1.12	0.2173	15.6637	72.09	200.90
Series 3	100.0x100.0x4.5	424.49	474.94	1.12	0.2106	16.0608	76.28	203.51
	120.0x120.0x3.0	400.77	512.76	1.28	0.1988	16.3567	82.29	203.65
	120.0x120.0x4.5	436.54	530.41	1.22	0.2139	15.1161	70.66	206.00
	150.0x150.0x3.0	429.14	503.86	1.17	0.2078	14.7328	70.90	208.52
	150.0x150.0x4.5	334.80	474.76	1.42	0.1689	18.7575	111.08	200.62

According to EN 1993-1-1 (EC3) [19] and SANS 10162-1 [18], plastic analysis may be utilised in the global analysis of structures or their elements provided that the steel complies with the following requirements:

1) The ratio of the specified ultimate tensile strength (f_u) to the specified yield strength (f_y) is at least 1.2 (EN 1993-1-1 [19] and SANS 10162-1 [18]).
2) The elongation at failure on a gauge length of $5.65\sqrt{A_o}$ in a tensile test is at least 15%, where A_o is the original cross-sectional area (EN 1993-1-1 [19] and SANS 10162-1 [18]).
3) The stress-strain diagram shows that the ultimate strain (ε_u) corresponding to the ultimate tensile strength (f_u) is at least 20 times the yield strain (ε_y) corresponding to the yield strength (f_y) (EN 1993-1-1 [19]).
4) The specified yield stress does not exceed 700 MPa for SANS 10162-1 [18] and 460MPa for EC4 [13] to promote ductility

The requirement of $f_u/f_y > 1.2$ is easily satisfied by the coupons from the larger and milder steel sections (150x150x3.5 SHSS in Series 1, and all the sections in Series 3, except 100.0x100.0x4.5 SHSS). SHSSs of high strength steel and smaller size sections (101.6x101.6x3.0 and 120.0x120.0x3.5 sections in Series 1, all Series 2 sections, and 100.0x100.0x4.5 in Series 3) achieved an ultimate strength to yield strength ratio, which is lower than the specified ratio. Most of the smaller size sections in Series 2 are less likely to be used structural elements and the high strength exhibited by these sections might be partly due to the rolling process. The elongation at failure on a gauge length of $5.65\sqrt{A_o}$ was not determined in these tests. Requirement 3 ($\varepsilon_u/\varepsilon_y > 20$) is easily satisfied by all the coupons of the SHSSs tested, except the 60.0x60.0x4.0 section in Series 2. This section achieved the largest yield strength and is the smallest SHSS size tested. All the coupons satisfies the requirement provided by SANS 10162-1 [18] in (4), however, the coupons from the 101.6x101.6x3.0 SHSS in Series 1 and half of the sections in Series 2 did not satisfy the more stringent requirement provided by EC4 [13]. Table 1 show that the values of $\varepsilon_u/\varepsilon_y$ increase with a reduction in yield strength. SHSSs with high values of $\varepsilon_u/\varepsilon_y$ are likely to be more ductile than those with corresponding low values.

2.2 Properties of concrete

Grade 30MPa concrete was specified for use in the composite columns, since it is the most readily available concrete in South Africa. To achieve this strength a concrete mix was prepared according to the specifications from the Cement and Concrete Institute [21]. This specification requires that the cement, sand, 19mm stone and water be proportioned according to the following ratio: 149.6kg, 268.0kg, 341.1kg and 73.1l respectively. A total number of 6 concrete cubes, 100x100x100mm, were cast from the same mix prepared for the composite columns for each series of tests. On the next day, these cubes were removed from the moulds and placed inside a curing bath for a period of 28 days prior to testing. The temperature inside the curing bath was kept at about 23°C.

Casting of concrete cubes and composite columns was done on the same day to ensure that the results are uniform after 28 days. The concrete cubes were tested according to SANS 5861-1 [22]. An Avery-Denison machine with a capacity of 2000kN was used to test the cubes and the average results are given in Table 2. The cubes were compressed until failure occurred by crushing of the cubes, and the ultimate load (P_u) was recorded. The ultimate load was then divided by the compressed area of the cube to evaluate its compressive strength. In order to apply EC4 [13], the cube strength of the concrete (f_{cu}) is converted into a cylinder strength (f_{cu}'), as required by the code. The unconfined concrete cylinder compressive strength (f_{cu}') is approximately equal to 0.8 times the unconfined concrete cube compressive strength ($0.8f_{cu}$). The modulus of elasticity of the concrete (E_c) was determined from Table 1 of SANS 10100-1 [23]. This table relates the cube strength of the concrete (f_{cu}) to its modulus of elasticity (E_c). SANS10100-1 [23] is based on the British concrete code, BS EN 1992-1-1 [24].

Table 2: Average cube test results

Series	Mass (kg)	Density (kg/m$^{3)}$	P_u (kN)	f_{cu} (MPa)	f_{cu}' (MPa)	E_c (GPa)
Series 1	2.271	2271	312	31.2	25.0	28.25
Series 2	2.419	2419	360	36.0	28.8	29.57
Series 3	2.419	2419	360	36.0	28.8	29.57

3. Testing programme

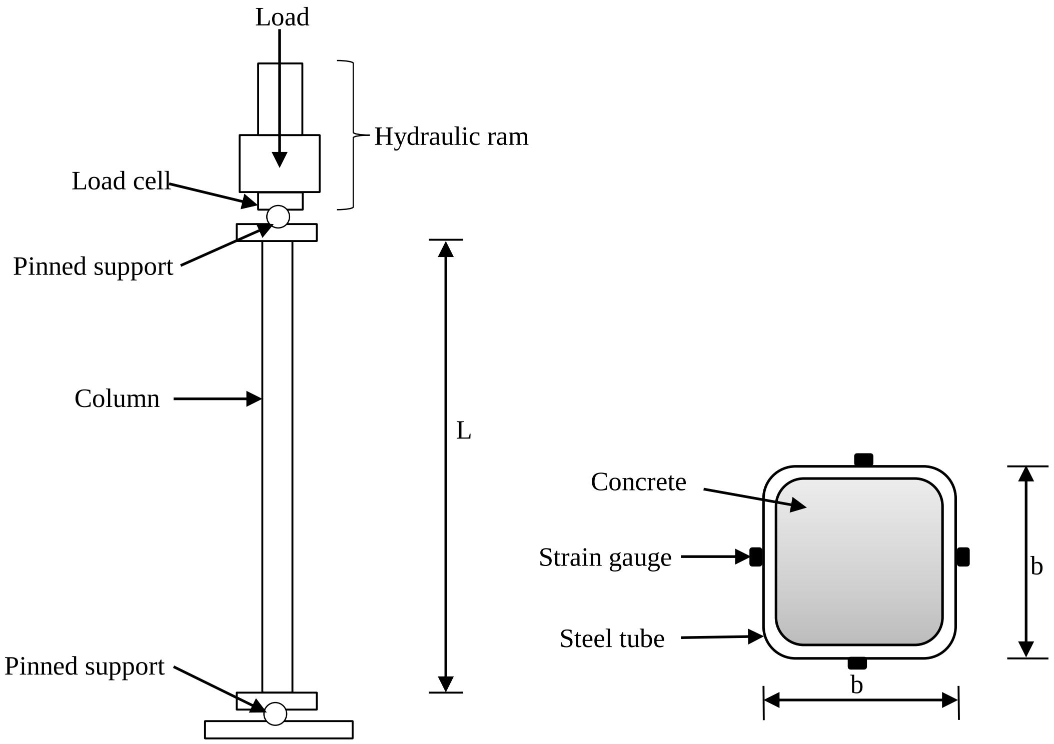
3.1 Specimen preparation

A total number of 29 square composite columns were prepared for testing. These columns varied in lengths and cross-sectional areas. Initially, the column lengths chosen for this investigation were 101.6x101.6x3.0mm, 120x120x3.5 mm and 150x150x3.5 mm SHSSs of 1.0 m, 1.5 m, 2.0 m and 2.5 m length only. This group of sections (Series 1) could not cover the scope of this investigation adequately. To increase the slenderness ratio of the columns, a decision was taken to include SHSSs, in Series 2 and 3 into the investigation. All columns were filled with grade 30 concrete. A 5mm thick end-plate was tack-welded to the bottom of each column to contain the concrete during casting. The concrete was placed in 5 layers and compacted with a vibrator to remove air pockets. The top part of the specimens was covered with plastic sheeting to prevent moisture from evaporating for a period of 28 days. This was done to ensure that the concrete achieves its 28-day strength. After 28 days, the plastic sheeting was removed from each column and the top surfaces were levelled with an angle grinder to ensure uniform load distribution over the top surface.

3.2 Test set-up and procedure

The Moog machine, with a capacity of 2500kN, was used to test Series 1 composite columns, the Instron machine, with a capacity of 500kN, was used to test Series 2 composite columns, and the Mohr and Federhaff compression machine, with a capacity of 9000kN, was used to test Series 3 composite columns. The load was applied concentrically through specially designed plates with circular grooves at the centre to accommodate a steel ball. Only one groove plate was used at the top of the specimens since the head of the machine had an in-built socket. This socket ensured that the load remained concentrically loaded during testing. The steel ball ensured that the applied load is a point load and that the plates, directly below and above the column, guaranteed that the load was applied simultaneously to the concrete core and the steel tube. This configuration also made it possible to have pinned end conditions in the column. The specimens were loaded at a rate of 2mm/min until failure to ensure that the behaviour was carefully observed. Figure 1 (a) shows a schematic diagram of the typical test set-up.

An in-built linear variable displacement transducer (LVDT) was used to measure axial shortenings as the load was applied. Strain gauges were attached at mid-height, on two opposite sides of each column, to measure circumferential surface strains (Figure 1(b)). Both strain gauge and LVDT readings were recorded on a computer connected to the machine.



(a) Test set-up (b) Location of strain gauges

Figure 1: Schematic test set-up and location of strain gauges

4. Failure modes

The general failure mode observed in the specimens was overall member buckling (OB). All the 1.5m to 2.7m columns failed largely by overall buckling (Figure 2(b)–(c)). Overall instability was promoted by the large slenderness ratio. The shorter columns, typically 1m of the larger sections (150x150x3.5), exhibited bulging (local buckling (LB)) behaviour at the centre and near the top end (Figure 2(a)). After cutting the cross-section of a few short SHSSs, it was observed that bulging was characterised by plastic flow of concrete to fill up the void, caused by outward buckling. Apart from overall member buckling, some intermediate length columns also displayed local buckling away from the centre, at about a quarter of the column length from the top of the column. This is evidence that overall flexural buckling was coupled with local instability, because of the large plate slenderness. In both columns the concrete forced the steel tube to buckle outwards. Specimens with high d/t ratios had more local buckling or bulging compared to those with small d/t ratios. A summary of the failure modes is given in Table 4.

(a) 1.0m (b) 1.5m (c) 2.0m (d) 2.5m (c) 2.7m

Figure 2: Typical failure modes of composite columns.

5. Test and theoretical results

5.1 Code provisions

SANS 10162-1 [18] and EC4 [13] prescribes different limits about the compressive strength of concrete, yield strength of steel and width-to-thickness ratio (d/t) that must be used in composite columns. SANS10162-1 is based on CAN/CSA-S16-09 [20]. SANS10162-1 [18] requires the concrete strength to range from 25MPa to 100MPa for axially loaded CFST columns and limits the specified yield stress (f_y) of the steel to 700MPa, to promote ductility. For the same reason, EC4 [13] specifies the characteristic concrete cylinder strength (f_{cu}') and the yield stress (f_y) of the steel to range from 20MPa to 50MPa, and 235MPa to 460MPa, respectively. As shown in Table 2, concrete requirements satisfy both SANS 10162-1 [18] and EC4 [13], however, all the steel sections in Table 1 satisfy SANS 10162-1 [18], and not EC3 [13]. New structures may require the use of high strength materials, thus it is necessary to extend beyond the limits of design specifications.

The slenderness and width-to-thickness ratios are given in Table 3, where parameters b, L and t represent the outside width, length and thickness of the square hollow steel sections. The slenderness ratio (L/d) of the composite column ranged from 6.67 to 45.00. This classifies the columns as short to slender columns. Intermediate and slender columns reflect the lengths that are expected in practice.

The width-to-thickness ratio (b/t) of the columns ranges from 13.33 to 50.00. SANS 10162-1 [18] and EC4 [13], recommends that the outside width-to-thickness ratios of square hollow structural sections in compression be limited to $b/t \leq 1350/\sqrt{f_y}$ and $b/t \leq 52\sqrt{235/f_y}$, respectively (Table 3). Although SANS10162-1 suggest that local buckling will not be the critical failure mode, a few sections did not meet EC4 local buckling requirements. These include the 150.0x150.0x3.5 section of 1 - 2.5m length, and 120.0x120.0x3.0 and 150.0x150.0x3.0 sections of 2.7m length. A comparison of the provision from these codes shows that EC4 [13] has more stringent requirements than SANS 10162-1 [18]. Residual stresses were not measured, however, initial-out-straightness of columns was measured and was found to be within the general limit of L/1000, allowed by SANS10162-1 [18] and EN 1993-1-1 [19]. EN 1994-1-1 [13] provides a relaxed member imperfection of L/300.

Table 3: Slenderness ratio and diameter-to-thickness

Series	SHSS	L (mm)	t (mm)	f_y (MPa)	L/b	b/t	Limit (b/t)	
							SANS	EC4
Series 1	101.6x101.6x3.0	1000	3.0	483.90	9.84	33.87	61.37	36.24
	101.6x101.6x3.0	1500	3.0	483.90	14.76	33.87	61.37	36.24
	101.6x101.6x3.0	2000	3.0	483.90	19.69	33.87	61.37	36.24
	101.6x101.6x3.0	2500	3.0	483.90	24.61	33.87	61.37	36.24
	120.0x120.0x3.5	1000	3.5	401.00	8.33	34.29	67.42	39.81
	120.0x120.0x3.5	1500	3.5	401.00	12.50	34.29	67.42	39.81
	120.0x120.0x3.5	2000	3.5	401.00	16.67	34.29	67.42	39.81
	120.0x120.0x3.5	2500	3.5	401.00	20.83	34.29	67.42	39.81
	150.0x150.0x3.5	1000	3.5	405.00	6.67	42.86	67.08	39.61
	150.0x150.0x3.5	1500	3.5	405.00	10.00	42.86	67.08	39.61
	150.0x150.0x3.5	2000	3.5	405.00	13.33	42.86	67.08	39.61
	150.0x150.0x3.5	2500	3.5	405.00	16.67	42.86	67.08	39.61
Series 2	60.0x60.0x3.0	2700	3.0	351.51	45.00	20.00	72.01	42.52
	60.0x60.0x4.0	2700	4.0	528.55	45.00	15.00	58.72	34.67
	60.0x60.0x4.5	2700	4.5	522.11	45.00	13.33	59.08	34.89
	63.0x63.0x3.0	2700	3.0	442.69	42.86	21.00	64.16	37.89
	63.0x63.0x4.0	2700	4.0	517.49	42.86	15.75	59.34	35.04
	63.0x63.0x4.5	2700	4.5	449.95	42.86	14.00	63.64	37.58
	76.0x76.0x3.0	2700	3.0	441.12	35.53	25.33	64.28	37.95
	76.0x76.0x4.0	2700	4.0	498.38	35.53	19.00	60.47	35.71
	76.0x76.0x4.5	2700	4.5	500.97	35.53	16.89	60.32	35.61
	100.0x100.0x3.0	2700	3.0	441.19	27.00	33.33	64.27	37.95
Series 3	100.0x100.0x4.5	2700	4.5	424.49	27.00	22.22	65.52	38.69
	120.0x120.0x3.0	2700	3.0	400.77	22.50	40.00	67.44	39.82
	120.0x120.0x4.5	2700	4.5	436.54	22.50	26.67	64.61	38.15
	150.0x150.0x3.0	2700	3.0	429.14	18.00	50.00	65.17	38.48
	150.0x150.0x4.5	2700	4.5	334.80	18.00	33.33	73.78	43.57

According to SANS10162-1 [18], the unfactored compressive resistance of square concrete-filled steel tubes is given by

$$N = \left(\tau A_s f_y + 0.68\tau' A_c f_{cu}\right)\left(1+\lambda^{2n}\right)^{-1/n} \qquad (1)$$

where, $n = 1.8$, steel reduction factor $\tau = 1$, concrete enhancement factor $\tau' = 1$ and the relative slenderness $\lambda = \sqrt{\left(N_p/N_{ec}\right)}$. The factor of 0.68 relates the strength obtained from a standard cube test to uniaxial strength. Since the steel reduction factor $\tau = 1$ and concrete enhancement factor $\tau' = 1$, this means that confinement effects in square concrete filled tubes are ignored. The parameter n reflects the difference in the patterns and levels of residual stress and initial out-of-straightness and N_p is equal to the compressive resistance (N) given by Equation 1 with a relative slenderness (λ) of zero. The elastic critical load is $N_{ec} = \pi^2 EI_e/(KL)^2$, where the effective elastic flexural stiffness is

$EI_e = E_s I_s + K_c E_c I_c / (1 + N_{us}/N_u)$. This stiffness is obtained by adding up the flexural stiffness of the individual components of the cross-section. In composite columns the value of the flexural stiffness of the composite column may decrease with time due to creep and shrinkage of concrete. Creep effects are accounted for by reducing the initial elastic modulus of the concrete as shown in the second term. Since the applied loading is short-term and the concrete core is essentially sealed from migration of any moisture due to presence of the tube, drying creep and shrinkage strains are considerably lower in concrete filled tubes. In addition multi-axial stress effect does not allow concrete to freely creep in the axial direction. Thus for short-term loading the effective elastic flexural stiffness simplifies to $EI_e = E_s I_s + K_c E_c I_c$, where the factor $K_c = 0.6$ is an empirical multiplier (determined by a calibration exercise to give good agreement with test results). The parameters I_s, I_c, E_c, N_{us} and N_u are the moment of inertia of the steel, moment of inertia of the concrete, elastic modulus of concrete, sustained load on the column and total axial load on the column, respectively.

In EC4 [13] the squash or plastic resistance is calculated using the simplified design method and is obtained by adding the plastic resistance of the two components as follows:

$$N_{pl,Rd} = A_s f_y + A_c f_{cu}' \tag{2}$$

where, A_s and A_c are the area of steel and concrete, and f_y and f_{cu}' are the yield strength of steel and cylinder compressive strength of concrete, respectively. As in Equation (1), Equation (2) also ignores confinement effects. The scope of this simplified method is limited to doubly symmetrical members and of uniform cross-section over its length. Equation 2 assumes that the structural steel and the concrete will fail by yielding and crushing, respectively [13]. The factor of 0.85 is replaced by 1.0 for concrete-filled sections because of better confining effect of the tube. The maximum strength of concrete is achieved due to the protection against the environment and against splitting of concrete. Clause 6.7.1(4) of EC4 [13] requires that the steel contribution ratio δ of the composite compression member be $0.2 \le \delta \le 0.9$ and the maximum non-dimensional slenderness ratio or relative slenderness of the composite column λ be limited to 2.0. If δ is less than 0.2 or greater than 0.9 then there is no composite action. In the case where δ is less than 0.2, the column should be designed as reinforced concrete only [23], and if δ is larger than 0.9, the concrete is ignored in the calculations, and the column is designed as a bare structural steel section [19]. The specimens investigated had a steel contribution ratio of 0.62-0.72 and a relative slenderness of 0.25-1.0, which confirms that the sections are indeed composite.

The plastic resistance defined by Eq. (2) represents the maximum load that can be applied to a short column. For slender columns with low elastic critical load, overall flexural buckling may be critical. The overall flexural buckling resistance of a concrete filled steel tube may be expressed as a proportion of the plastic resistance, as given in Equation 3.

$$N = \chi N_{pl,Rd} \tag{3}$$

where, χ is a reduction factor. The reduction factor is calculated using European strut curves and is given by $\chi = 1/\left(\phi + \sqrt{\phi^2 - \lambda^2}\right)$, where $\phi = 0.5\left(1 + \alpha(\lambda - 0.2) + \lambda^2\right)$ and the imperfection factor for hollow sections is $\alpha = 0.21$. Note that $\chi \le 1.0$. In EC4 [13] the relative slenderness or non-dimensional slenderness of the column is defined by $\lambda = \sqrt{N_{pl.Rd}/N_{cr}}$. $N_{pl.Rd}$ is equal to the compressive resistance given by Equation 2 and the elastic critical load for the column length, N_{cr}, is calculated from $N_{cr} = \pi^2 (EI)_e / L^2$, where the effective elastic flexural stiffness $(EI)_e = E_s I_s + K_c E_c I_c$, as given in SANS10162-1 [18].

5.2. Analysis Results

Table 4 shows a comparison of the experimental results of square concrete-filled steel tube columns with the predicted compressive resistance, calculated using SANS10162-1 [18] and EC4 [13]. In this table N_{TEST} is the maximum applied load to the concrete filled SHSS, N_{SANS} is the compressive resistance concrete filled SHSS, calculated using SANS10162-1 [18] and N_{EC4} is the compressive resistance of the concrete filled SHSS, calculated using EC4 [13]. It should be noted that both codes do not consider confinement in determining the compressive resistance of square CFSTs. As expected the compressive resistance of all columns decreases with increase in length of columns (Table 4).

Table 4: Test and calculated resistance of SCFT columns

Series	SHSS	L/b	b/t	λ	f_y (MPa)	N_{TEST} (kN)	N_{SANS} (kN)	N_{EC4} (kN)	N_{TEST}/N_{SANS}	N_{TEST}/N_{EC4}	Failure Modes
Series 1	101.6x101.6x3.0	9.84	33.87	0.39	483.9	718.88	752.5	763.2	0.96	0.94	OB
	101.6x101.6x3.0	14.76	33.87	0.58	483.9	691.38	711.0	713.3	0.97	0.97	OB
	101.6x101.6x3.0	19.69	33.87	0.78	483.9	589.48	633.8	638.6	0.93	0.92	OB
	101.6x101.6x3.0	24.61	33.87	0.97	483.9	471.29	534.9	535.2	0.88	0.88	OB
	120.0x120.0x3.5	8.33	34.29	0.31	401.0	1164.16	917.7	947.3	1.27	1.23	OB+LB
	120.0x120.0x3.5	12.50	34.29	0.46	401.0	997.19	894.7	907.0	1.11	1.10	OB
	120.0x120.0x3.5	16.67	34.29	0.61	401.0	899.65	846.2	854.1	1.06	1.05	OB
	120.0x120.0x3.5	20.83	34.29	0.77	401.0	729.10	770.9	780.4	0.95	0.93	OB
	150.0x150.0x3.5	6.67	42.86	0.25	405.0	1516.26	1259.9	1324.7	1.20	1.14	OB+LB
	150.0x150.0x3.5	10.00	42.86	0.37	405.0	1330.13	1245.0	1283.8	1.07	1.04	OB
	150.0x150.0x3.5	13.33	42.86	0.50	405.0	1307.61	1211.7	1235.2	1.08	1.06	OB
	150.0x150.0x3.5	16.67	42.86	0.62	405.0	1198.25	1154.4	1173.6	1.04	1.02	OB
Series 2	60.0x60.0x3.0	45.00	20.00	1.61	351.51	105.4	110.0	103.5	0.96	1.02	OB
	60.0x60.0x4.0	45.00	15.00	1.94	528.55	127.7	136.6	127.5	0.93	1.00	OB
	60.0x60.0x4.5	45.00	13.33	1.93	522.11	156.0	149.9	139.9	1.04	1.12	OB
	63.0x63.0x3.0	42.86	21.00	1.69	442.69	123.6	129.2	121.2	0.96	1.02	OB
	63.0x63.0x4.0	42.86	15.75	1.81	517.49	154.7	160.9	150.3	0.96	1.03	OB
	63.0x63.0x4.5	42.86	14.00	1.70	449.95	168.6	175.2	164.1	0.96	1.03	OB
	76.0x76.0x3.0	35.53	25.33	1.39	441.12	212.0	226.4	216.1	0.94	0.98	OB
	76.0x76.0x4.0	35.53	19.00	1.46	498.38	247.3	284.2	269.3	0.87	0.92	OB
	76.0x76.0x4.5	35.53	16.89	1.47	500.97	266.5	308.5	291.9	0.86	0.91	OB
	100.0x100.0x3.0	27.00	33.33	1.06	441.19	405.0	469.0	466.6	0.86	0.87	OB
Series 3	100.0x100.0x4.5	27.00	22.22	1.04	424.49	501.71	608.3	603.1	0.82	0.83	OB
	120.0x120.0x3.0	22.50	40.00	0.85	400.77	825.16	689.8	703.0	1.20	1.17	OB
	120.0x120.0x3.0	22.50	40.00	0.85	400.77	715.61	689.8	703.0	1.04	1.02	OB
	120.0x120.0x4.5	22.50	26.67	0.87	436.54	830.0	929.4	935.6	0.89	0.89	OB
	150.0x150.0x3.0	18.00	50.00	0.69	429.14	1087.1	1111.3	1139.4	0.98	0.95	OB
	150.0x150.0x4.5	18.00	33.33	0.64	334.80	1269.8	1230.7	1253.7	1.03	1.01	OB
	150.0x150.0x4.5	18.00	33.33	0.64	334.80	1300.4	1230.7	1253.7	1.06	1.04	OB

Generally, it can be observed from Table 4 that the strengths predicted by SANS 10162-1 [18] are almost the same with those predicted by EC4 [13]. The result of this is that the ratios of the test strength to the code values are also the same. In Series 1, with three groups of SHSS sections (101.6x101.6x3.0, 120.0x120.0x3.5 and 150.0x150.0x3.5) and of varying lengths (1.0 – 2.5m), both codes overestimate the capacity of columns with high strength steel and low-to-intermediate non-dimension slenderness ratio. This was also noticed in some sections in Series 2, such as 76.0x76.0x4.0, 76.0x76.0x4.5 and 100.0x100.0x3.0 SHSS sections, and in Series 3, such as 100.0x100.0x4.5 and 120.0x120.0x4.5 SHSS sections. For the 101.6x101.6x3.0 section of 1m length, both standards overestimate the test values by an average of 5%. This percentage increases to 12%

for a 2.5m length of the same cross-sectional size. On average, both codes overestimated the composite column capacity of this section by 7%.

In the case of the 120x120x3.5 section the two codes underestimate the 1m column by 23-25%. For the same length, the codes also underestimated the 150x150x3.5 section by 9-15%. This can be interpreted to imply that that stockier sections of larger d/t ratios possesses some confinement. A previous study found the corner regions in square concrete filled tubes to generate some confinement, however, the confinement quickly diminished beyond the corners and vanished completely at the center of the side walls [25]. Thus the confining pressure is not sufficient to overcome the effect of concrete degradation. Although the codes could not predict the strength of the 1m column accurately enough, they were able to predict the other lengths of the 120x120x3.5 and 150x150x3.5 sections reasonably well. Series 2 and 3 tests were included to widen the scope of this work to include intermediate-to-slender columns. All the columns in these groups are much slender than in Series 1 tests, and failed by overall flexural buckling. No evidence of confinement is observed in this group. The wall of the square tube resisted the concrete pressure by plate bending, instead of the membrane-type hoop stresses [26].

A clear trend is also shown in Table 4; the ratio of the test strength to the predicted strength decreases with increase in slenderness ratio. A probably cause of this is that the confinement effect is reduced with increasing length of the column, since the lateral deflection prior to failure increases the bending moment and reduces the mean compressive strain in the concrete. This trend means that both codes are not able to predict the composite columns of different length or slenderness ratio at the same level of accuracy. Stockier test columns tend to have larger strength than slender sections. For all the columns tested, EC4 predicted the compressive resistance better than SANS 10162-1. SANS 10162-1 tend to overestimate the strengths of columns.

A plot of the test strength as a function of code predicted loads is shown in Figure 3. In this figure, the points above the line are regarded as safe and those below the line are regarded as unsafe. As can be seen from the figure, all points for the 101.6x101.6x3.0 are on the unsafe side. The figure also shows that the codes tend to overestimate the test strength of short columns.

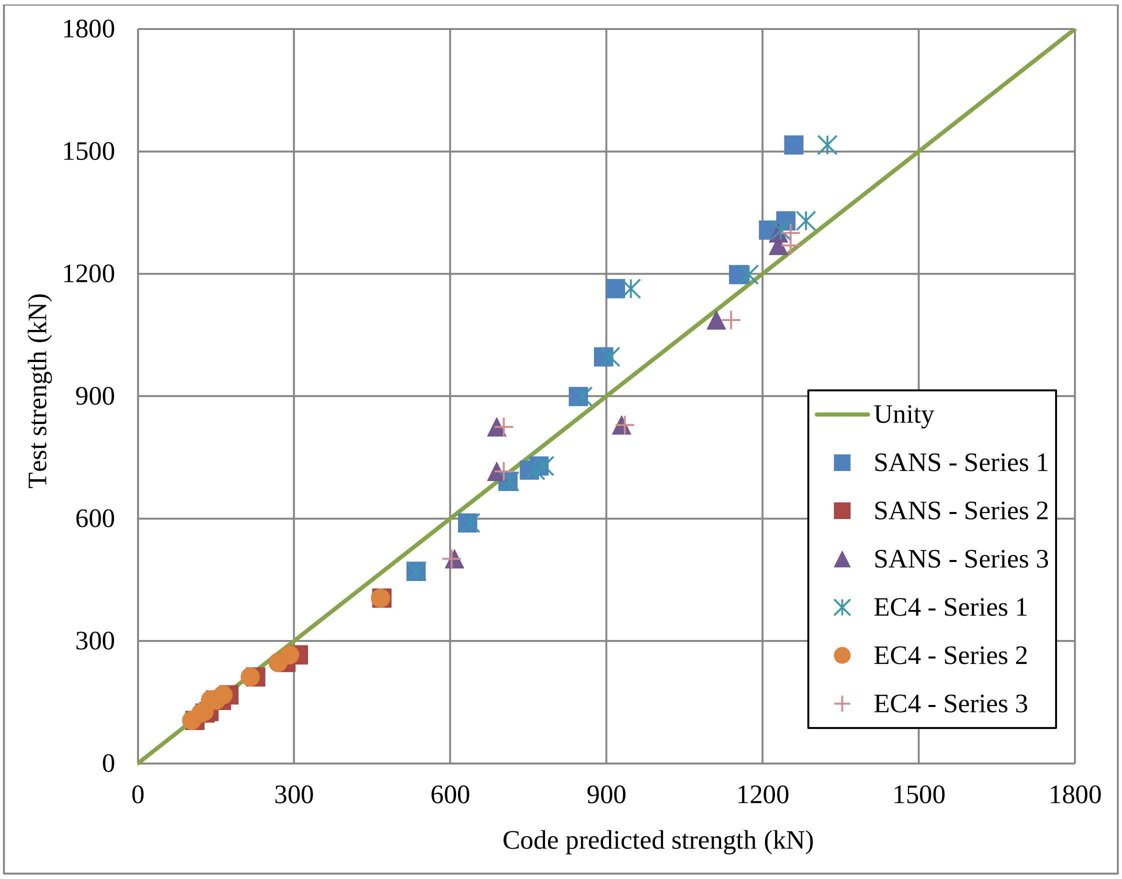


Figure 3 Test vs code predicted strength.

The effect of the non-dimensional slenderness ratio (λ) on the capacity of CFST columns is shown in Figure 4. In this figure, N_y is the squash or plastic resistance of the composite columns, NSANS (n = 1.8) is the compressive resistance curve, generated from SANS10162-1 [18] (Equation 1) and NSANS (n = 1.28) is the proposed curve. The other points in the figure represent the test results for the different sections. As expected, the compressive resistances of the composite columns decrease with the increases of the non-dimensional slenderness ratio. Since the compressive resistance results, obtained using the South African standard (SANS10162-1 [18]) were found to be very close to those obtained using EC4 [13] (Table 4 and Figure 3), a decision was taken in this paper to base the development of any new compressive resistance formula of square concrete-filled steel tubes on SANS10162-1 [18]. It is clear in Figure 4 that the proposed curve, models the strength of intermediate and slender columns far much better than the current curve, however, it underestimates the strength of shorter columns. Composite columns of intermediate length reflect the majority of compression members in steel structures and will buckle inelastically because of the compressive residual stresses on some parts of the cross-section and initial imperfections or out-of-straightness of the member.

To deal with this problem it is proposed to have a two-stage equation; one that models the results of short columns (Equation 4) and another one that models intermediate to slender columns (Equation 5).

$$N = A_s f_y + 0.68 A_c f_{cu} \qquad \text{for} \qquad \lambda \le 0.4; \qquad (4)$$

and

$$N = \left(A_s f_y + 0.68 A_c f_{cu}\right)\left(1 + \lambda^{2n}\right)^{-1/n} \qquad \text{for} \qquad \lambda > 0.4; \qquad (5)$$

where, n = 1.28. The experimental compressive results of short columns for the 120.0x120.0x3.5 and 150.0x150x3.5 are larger than the code predicted compressive results. This is caused by the confinement of the concrete, generated by the corners in the larger steel sections. Since the confinement of concrete in square sections is generally not as significant or reliable as in circular hollow sections, the crushing compressive resistance short columns (Equation 4) is expressed without any concrete enhancement or steel reduction. As shown in Equation 5 and Figure 4, the compressive resistance of intermediate and slender columns is less than the squash load and their capacity depends largely on the degree of slenderness ratio. The difference between the Equation 1 and 5 lies in the value of n. A value of n = 1.28 was found to model the test compressive resistance results far much better than when the value of n = 1.8.

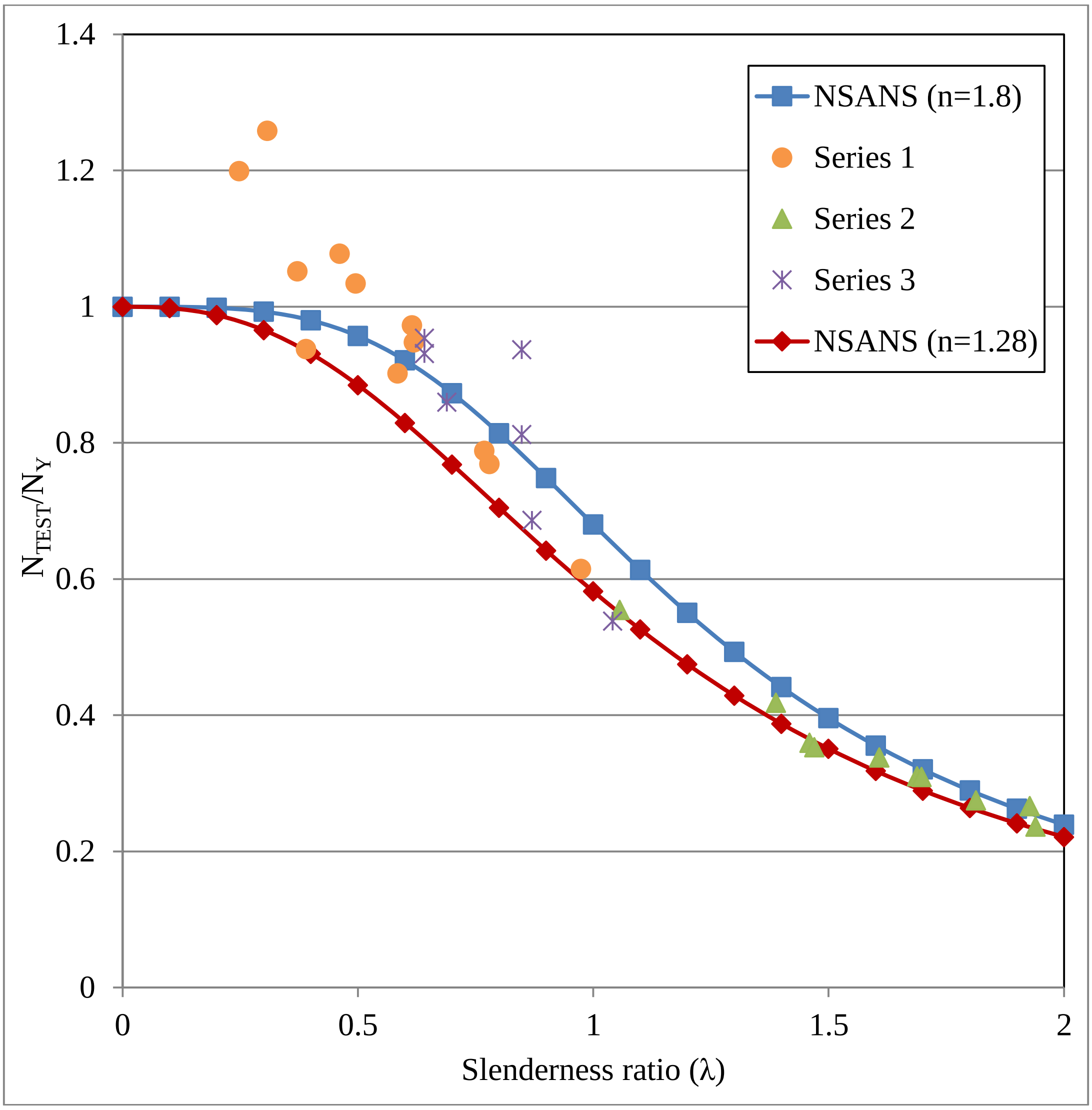


Figure 4 Load-slenderness ratio relationships

6. Load-axial shortening

Figures 5 to 7 show the relationship between the axial load applied to the columns and the corresponding axial shortening. The axial shortening of the columns was measured at pre-determined intervals, using an Instron built-in LVDT. The stiffness variation at low loads was caused by the uneven surface at the top of the column. The graphs are largely linear-elastic, followed progressively by an inelastic portion, as the steel started yielding. From these graphs it can be seen that the shorter CFST columns has a higher stiffness and load carrying capacity than the longer columns. Generally, the stiffness decreases with increase in length of all columns. For constant compressive strength of the concrete, and constant thickness and yield stress of the circular hollow section, the stiffness increases with increase in the diameter of the CFST columns. As the length of the composite columns increases, the load carrying capacity decreases. Composite columns with larger widths and steel thicknesses exhibited far much more inelasticity than the composite columns with smaller diameters and thicknesses. This is shown by a relatively flat graph at ultimate load, implying that local buckling occurred after achieving the yield strength (see Table 4 and Figure 7). The graphs also show that thicker tubes exhibited higher axial deformations just before local wall buckling. Smaller sized sections rapidly shedded their load once the maximum load was attained (Figure 6 and 7). Once the composite column has reached the yield value the square tube walls fail to provide enough confinement of the concrete core.

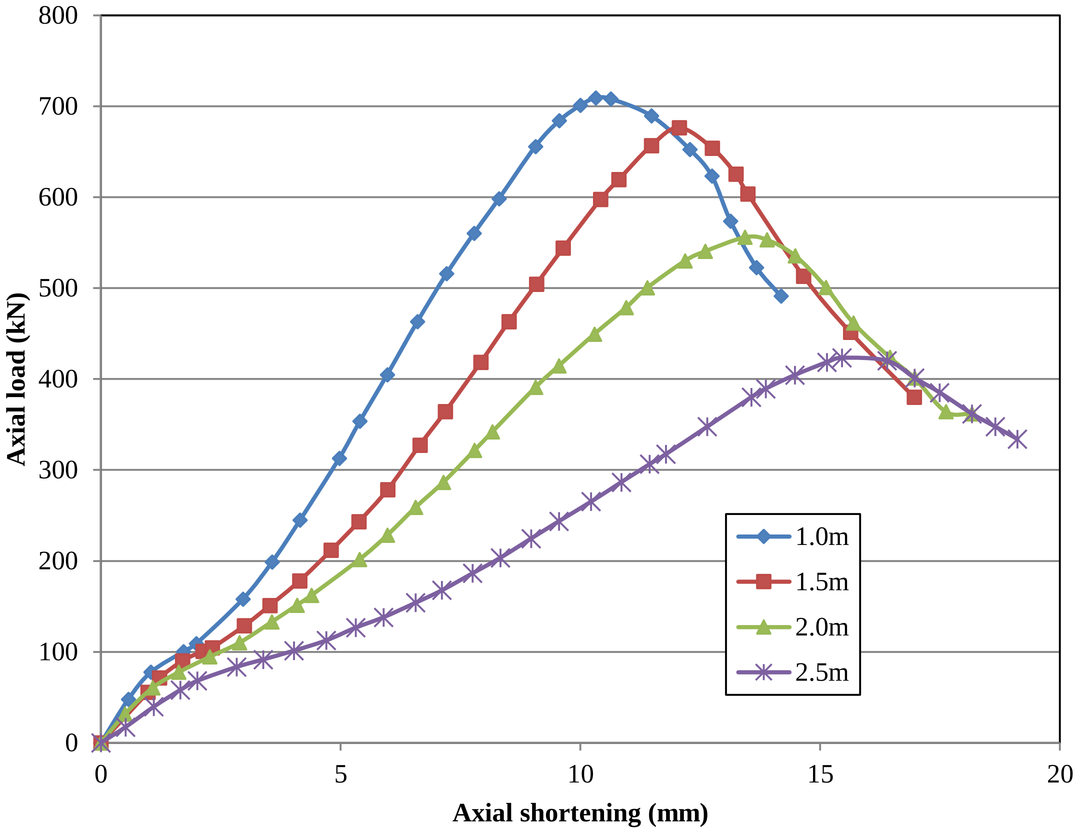


Figure 5: Axial load-shortening curves for 101.6x101.6x3.0mm composite columns

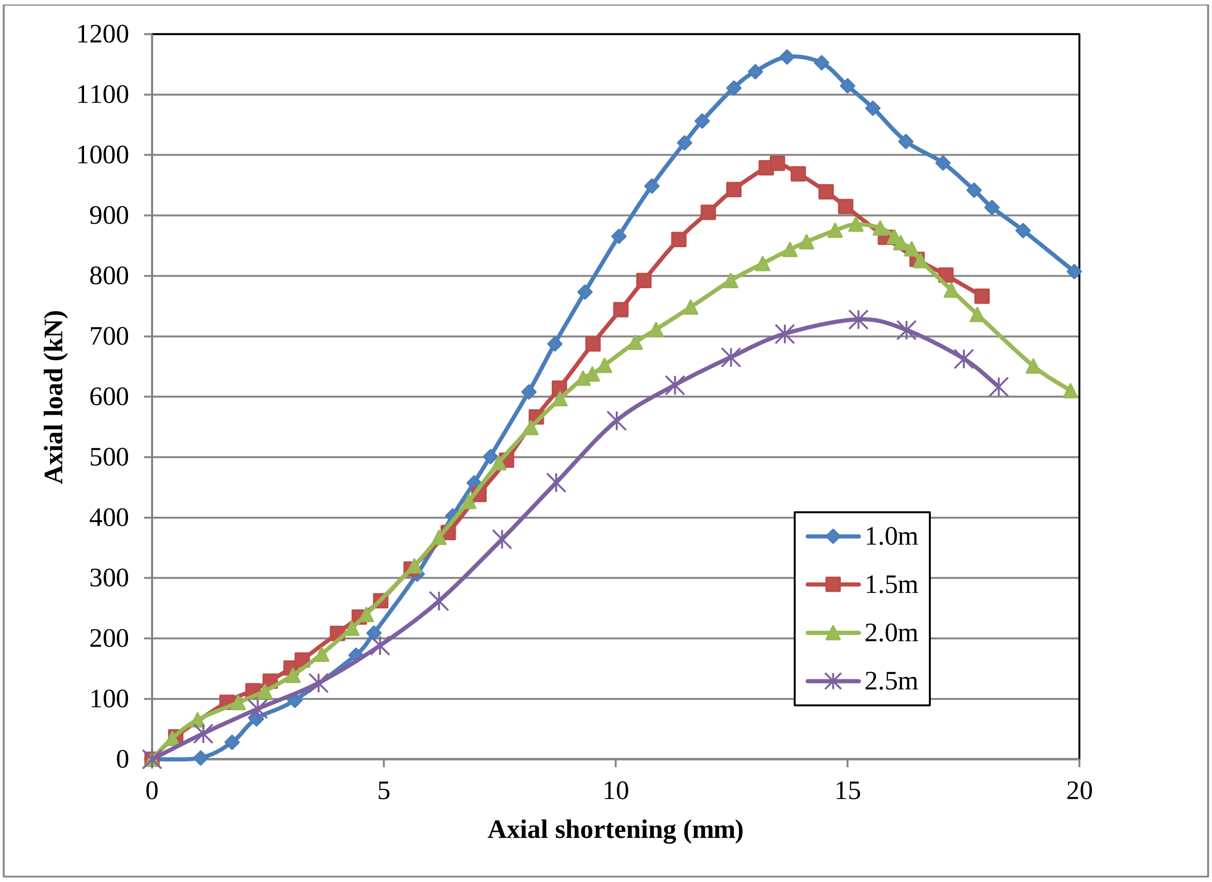


Figure 6: Axial load-shortening curves for 120x120x3.5mm composite columns.

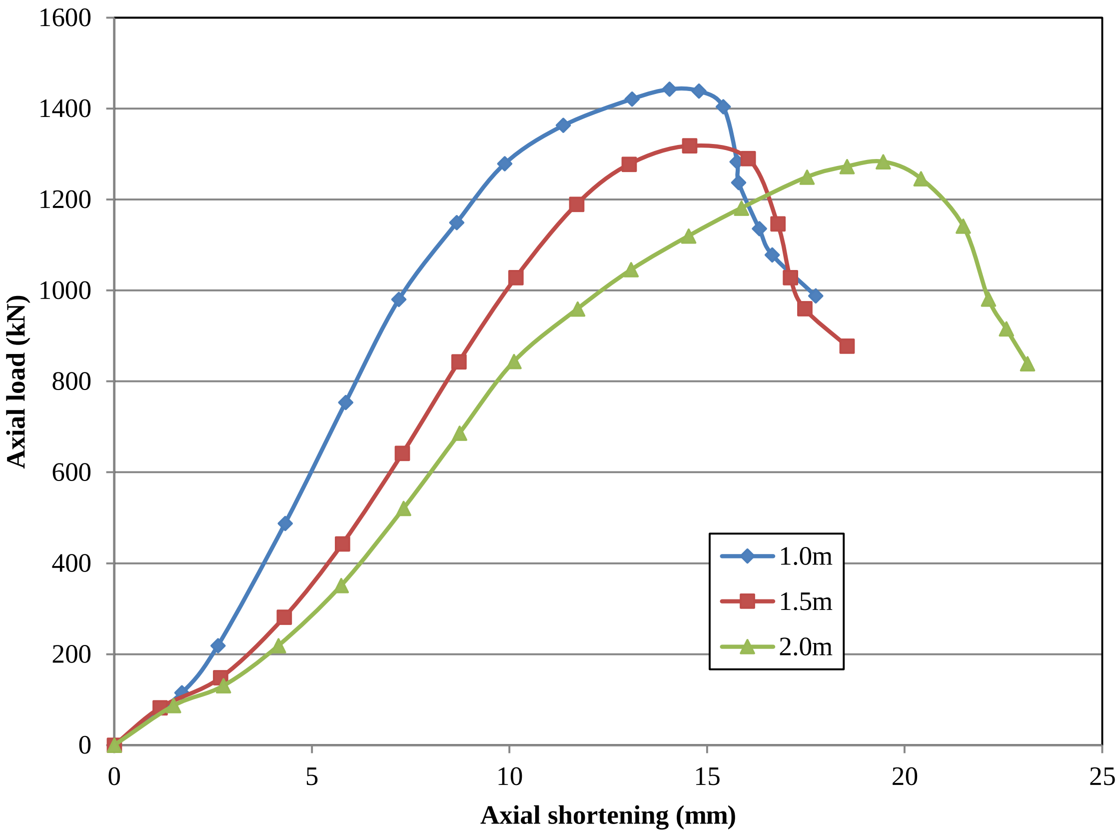


Figure 7: Axial load-shortening curves for 150x150x3.5mm composite columns.

7. Load - circumferential strain relationship of stockier columns

As shown in Table 4 and Figure 4, Series 1 columns can be classified as short-to-intermediate length columns. Such columns are expected to generate some circumferential stresses in SHSSs. Maximum circumferential expansion is expected to occur at mid-height of the steel tube walls, and this was measured by strain gauges attached to four columns faces of each specimen (Figure 1(a)). The relationship between axial load and circumferential strain of two critical faces is shown in Figures 8 to 10, and are useful in determining the onset of yielding, as well as highlighting local buckling on the column faces. It should be noted that for each section size, the strains are reported in terms of column length. The difference in signs for the strains indicates the way the specimens buckled with respect to the position of strain gauges. Before yielding of the steel takes place, the load-strain relationships for both graphs are linear and tensile (positive). In fact there was little or no difference between the strains, measured at two opposite locations of the cross-section. This is clear evidence that the column was subjected to uniform compression.

At the onset of overall buckling, all strains of the 101.6x101.6x3.0mm composite columns (Figure 8), recorded by Strain gauge 1 (SG1) reversed from tension to compression, however, Strain gauge 2 remained positive until failure. Significant reversal in strain is experienced in shorter columns compared to intermediate length members. As the size of the section increases, the number of reversed strains decreases. In the 120x120x3.5 composite columns (Figure 9), Strain gauge 1 for the 1.0 member length did not experience a change in sign, however, three strain gauges for member lengths 1.5m, 2.0m and 2.5m reversed. The number of strain gauges that did not undergo any strain reversal increased to two (1.0m and 1.5) for the 150x150x3.5 composite columns (Figure 10). This is clear evidence that all 101x101x3 sections failed by overall buckling only, whilst the 120x120x3.5 and 150x150x3.5 stockier sections experienced both overall and local buckling. Note that SG1 was swamped with SG2 in all the 1.5m length columns.

Failure of the composite column occurred after extensive yielding of the tensile face of the column section, as indicated by Strain gauge 2. This face provided a very smooth and ductile load-circumferential strain curves, which suggest a high deformation capacity. This probably explains the source of inelasticity in the sections tested. Strain gauge 1 curves show very sharp transitions from pre-peak to post-peak region and a sudden shedding of the load. To confine the concrete equal ductility must be achieved by both strain gauges, which is not the case with graphs in Figures 8-9. As shown in the graphs, larger sections more inelastic. The circumferential strains recorded in the investigation were relatively small. This is a result of local buckling which took place due to lack of confining pressure after yielding has occurred.

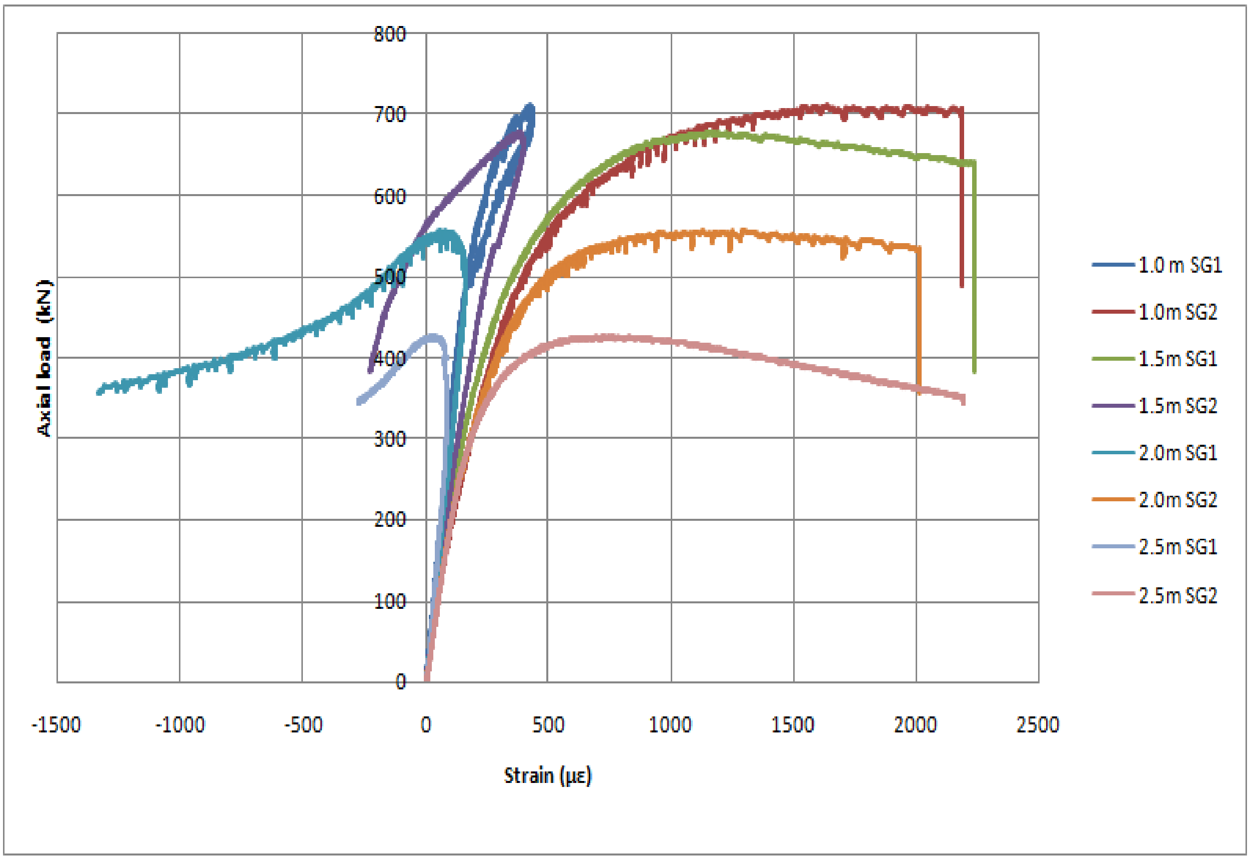


Figure 8: Load - circumferential strain curves for 101.6×101.6x3.0mm composite columns.

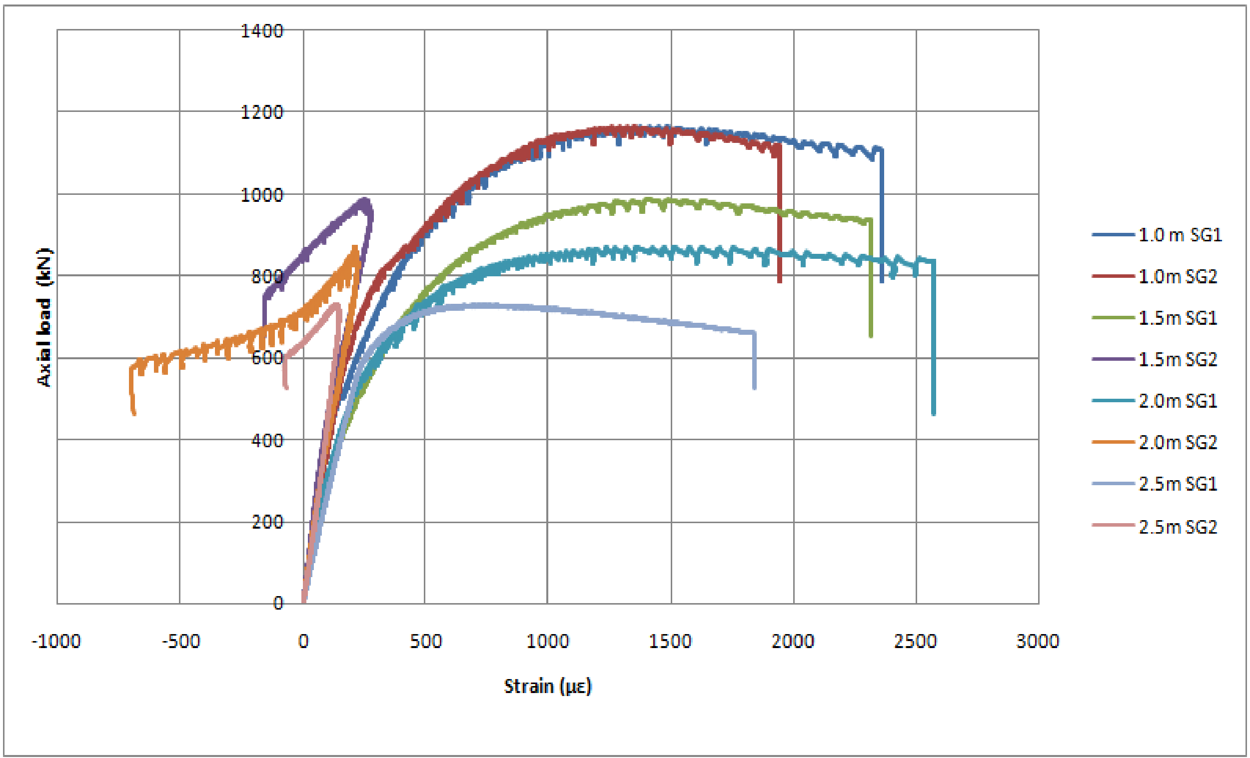


Figure 9: Load - circumferential strain curves for 120.0×120.0x3.5mm composite columns

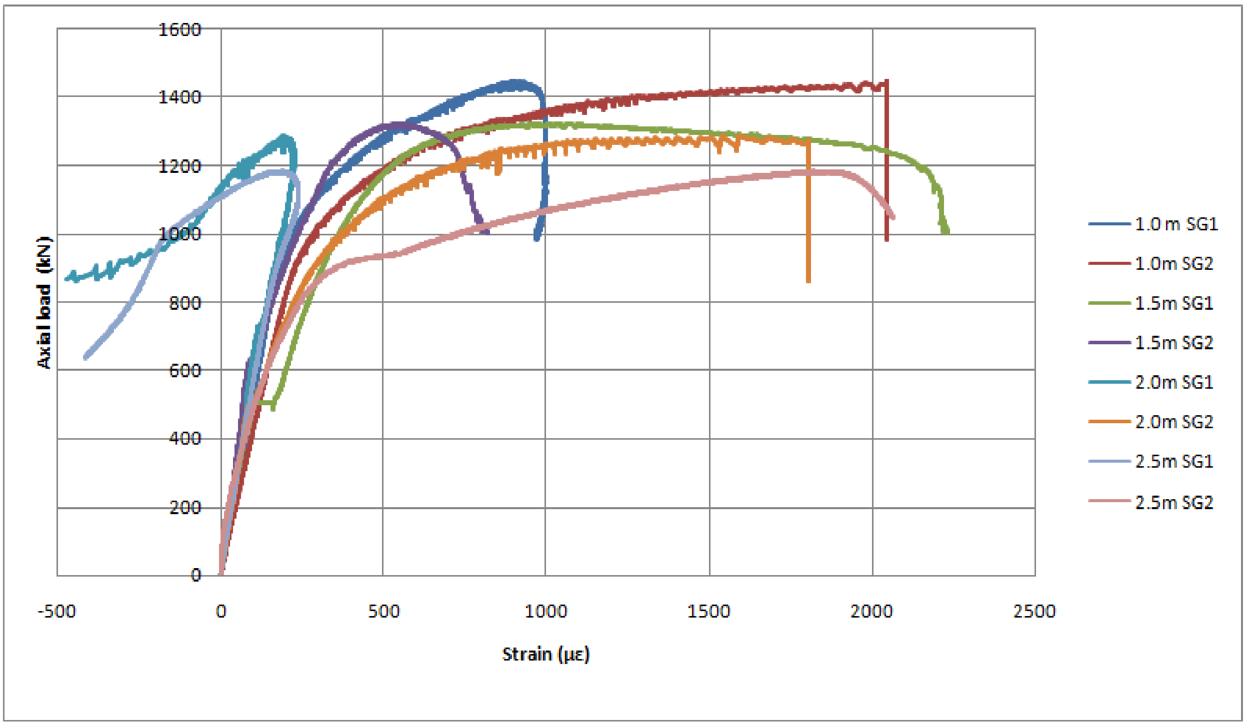


Figure 10: Load - circumferential strain curves for 150.0×150.0x3.5mm composite columns.

8. Load-longitudinal strain relationship of slender columns

Except one (100x100x3mm), all columns in Series 2 can be classified as slender columns. Such columns are not expected to generate any circumferential stresses in the SHSSs, but longitudinal stresses only. Similarly to circumferential stresses, longitudinal strain was measured at the mid-height of the column. As illustrated in Figures 11-13, both strain gauges (Strain Gauge 1 and 2) record the same compressive (negative) strain during the initial stages of loading, and the graphs are linear. In this graphs, the strains are reported in terms of the thickness of the section. Overall flexural buckling is noticed when the strain of Strain Gauge 2 (SG2) initially starts to decrease, and finally becomes tensile. As soon as the maximum strength of the column is reached, the columns started shedding the load. Smaller steel sections (60x60mm) shedded the load more than larger sections (76x76mm), and in all the graphs the transition of Strain Gauge 2 (SG2) from pre-peak region to post-peak region was sharper than Strain Gauge 1 (SG2). In each group (60x60mm, 63x63mm, and 76x76mm) thinner steel sections tend to shed the load more than thicker sections. Although failure of the composite column occurred without noticeable yielding of the steel section, significant inelasticity was noted in SG1 of 76x76x4.5.

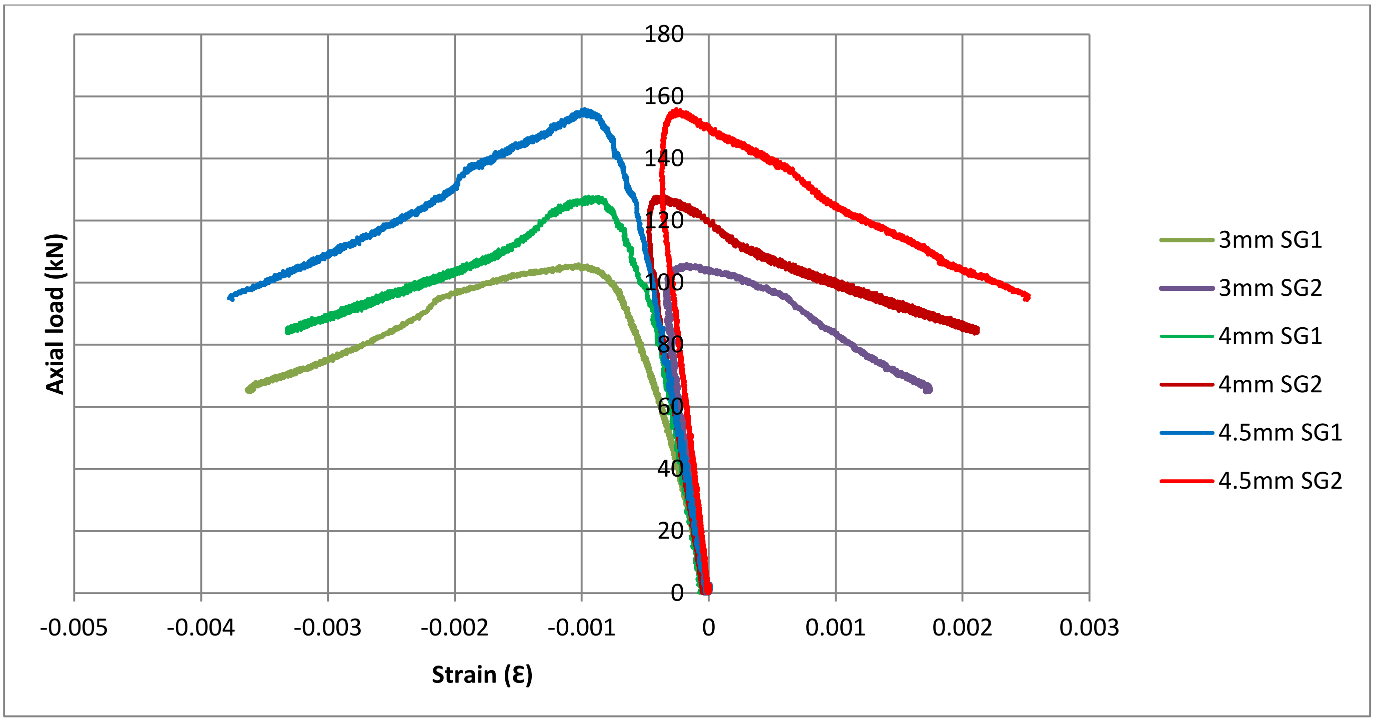


Figure 11: Load - longitudinal strain curves for the 60x60mm composite columns.

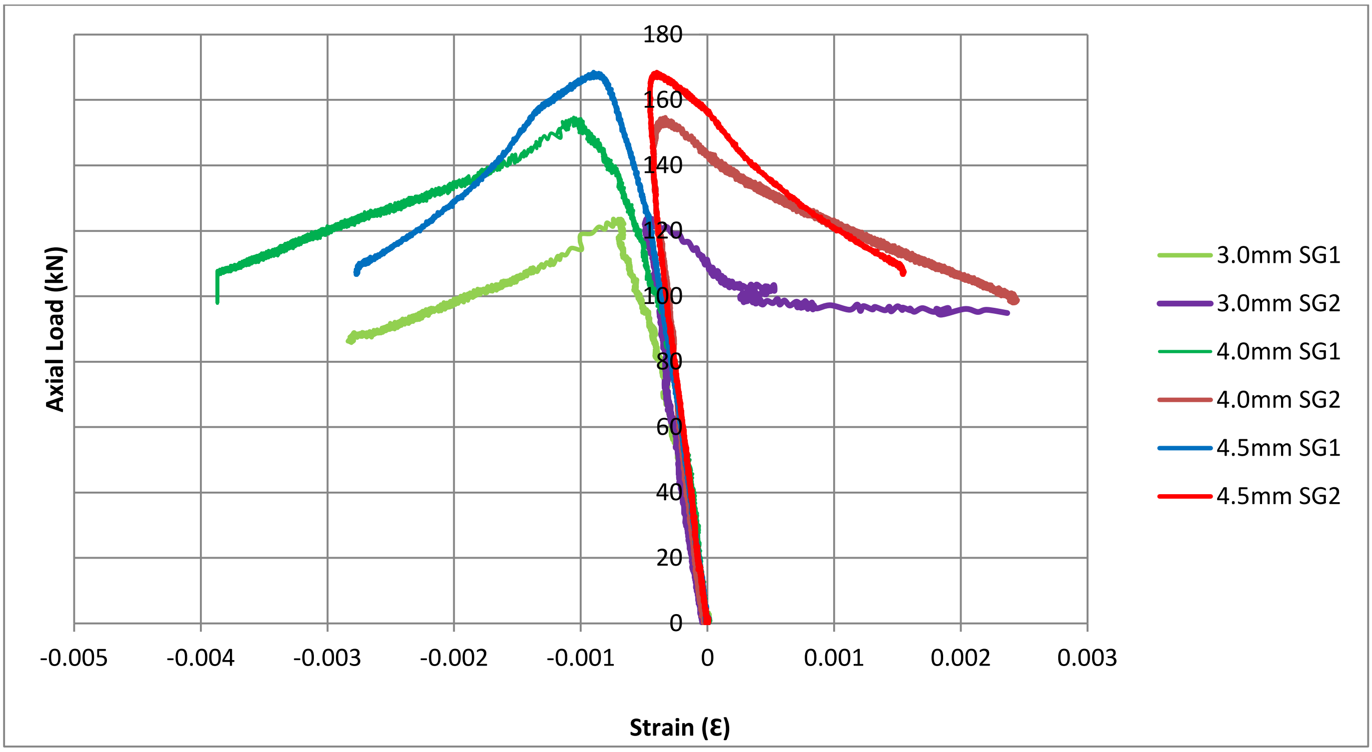


Figure 12: Load - longitudinal strain curves for the 63x63mm composite columns.

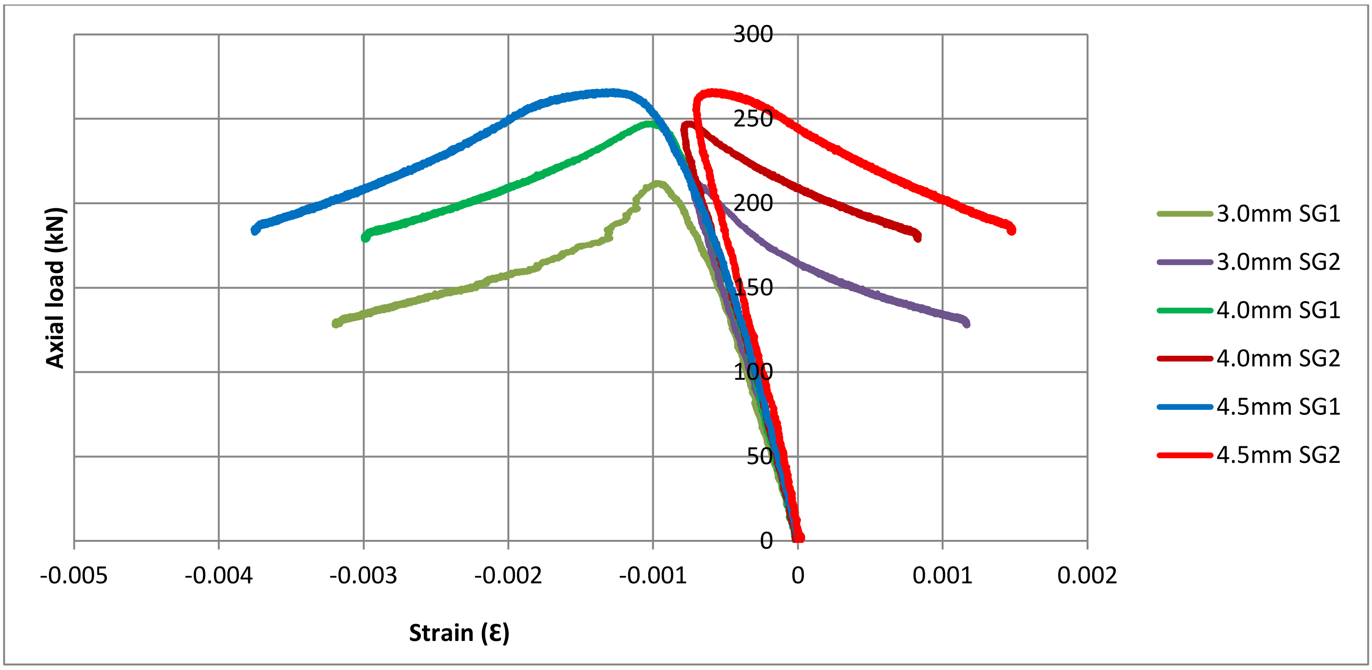


Figure 13: Load - longitudinal strain curves for the 76x76mm composite columns.

9. Conclusion

The failure mode observed in most specimens was overall member buckling. Overall buckling was observed in 1.5m to 2.7m length columns, whilst the shorter columns (typically 1m of larger sections (150x150x3.5)) experienced both overall buckling of the member and local buckling at the centre and near the top end. Specimens with high b/t ratios had more local buckling or bulging compared to those with small b/t ratios. All square concrete-filled tubes experienced a reduction in strength with an increase in slenderness ratio of the column. The load-axial deflection curves showed that all stockier specimens behaved in an inelastic manner, while the slender specimens behaved in an elastic manner.

Both codes overestimated the strengths of 101.6x101.6x3 square composite columns. On average, the codes overestimated the capacity of these composite columns by 7%. The two codes underestimated the stockier members of the 120x120x3.5 and 150x150x3.5 sections by about 25% and 17%, respectively. These sections were found to possess some confinement; however, the confining pressure is not sufficient to overcome the effect of concrete degradation. Both codes overestimate the capacity of columns with high strength steel and low-to-intermediate non-dimension slenderness ratio. A two-stage equation is proposed to model the results of short columns (Equation 4) and intermediate to slender columns (Equation 5).

Circumferential strains recorded at mid-heights of the specimens were very small. As a result, it is concluded that square concrete-filled steel tubes do not provide confinement of the concrete core. The change in sign of the strains recorded by Strain gauge 1 is important in determining the final mode of failure of the composite columns. When the final mode of failure is overall buckling only, the sign of Strain gauge 1 changes from positive to negative. However, when there is a combination local and overall buckling the sign of Stain gauge 1 remains positive.

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