Construction Materials and Structures
S.O. Ekolu et al. (Eds.)
IOS Press, 2014

doi:10.3233/978-1-61499-466-4-1148

Cold-formed circular hollow sections under axial compression

Morgan DUNDU[1] and Vongani CHABALALA
Department of Civil Engineering Science, University of Johannesburg, South Africa

Abstract. Two series of experiments were carried out to investigate the behaviour of pinned-ended circular hollow section (CHS) columns, subjected to axial compressive load. A total of 30 columns were tested in this investigation; 20 columns in Series 1 and 10 columns in Series 2. The outside diameter-to-thickness ratio (d/t) and the slenderness ratio (KL/r) ranged from 29.7 to 46.4 and 20.8 to 82.2 for Series 1, respectively, and from 55.0 to 62.9 and 10.7 to 34.9 for Series 2, respectively. In general, Series 1 columns failed by overall flexural buckling and, whilst Series 2 columns failed by local ring-type buckling. The test strengths of the columns were compared with the strengths predicted by the South African design standard (SANS10162-1) and the European design standard (EN 1993-1-1).

Keywords. Circular hollow sections, columns, diameter-to-thickness ratio, slenderness ratio, overall flexural buckling, local buckling.

Introduction

The behavior of axially loaded circular hollow steel sections (CHSs) has been an area of extensive research over a long period of time. However, not much study has been performed on thinner cold-rolled carbon sections which are in use today. To fill this gap hot-rolled column curves and design formulae have been used to define the behavior of thin cold-rolled sections. Although both materials are made out of carbon steel and possess a bilinear stress-strain relationship, thin cold-rolled sections do not have a well-defined yield plateau and are affected by local buckling far much more than hot-rolled sections. In addition, the amount and distribution of residual stresses, exhibited by the thin cold-rolled sections, are completely different from hot-rolled sections. The aim of this paper is to study the behaviour of these sections and compare the experimental strengths of the columns with the design strengths predicted by the South African standard, SANS 10162-1 [1], and the European standard, EN 1993-1-1 [2]. SANS10162-1 [1] is based on the Canadian code, CAN/CSA-S16-09 [3], and any reference to SANS10162-1 [1] also refers to CAN/CSA-S16-09 [3].

1. Material properties

In order to determine the material properties of the CHSs, tensile tests were conducted on the coupons. A total of 45 coupons were machined from the lengths supplied (3

[1]Corresponding author: University of Johannesburg, Department of Civil Engineering Science, P.O. Box 524, Auckland Park, 2006, South Africa; E-mail. mdundu@uj.ac.za

coupons from each length) and tested according to the guidelines provided by European Standard, EN ISO 6892-1 [4]. The coupons were tested using the 100kN capacity Instron 1195 tensile testing machine, at a rate of 3mm/min. A calibrated extensometer was attached to each coupon during testing to measure the longitudinal strain of the coupons. The average values of the yield stress (f_y), tensile strength (f_u) and Young's modulus of elasticity (E) for the tested coupons are presented in Table 1. Since the tested carbon steel did not have a defined yield point, the yield stress (f_y) of each coupon was determined from a 0.2% off-set line, parallel to the initial stress-strain curve. The average values of the yield stress (f_y) and Young's modulus of elasticity (E) were used to predict the strengths of the columns using SANS 10162-1 [1] and EN 1993-1-1 [2].

Table 1. Average material properties

Series	Section	Length (m)	f_y (MPa)	f_u (MPa)	E (MPa)
Series 1	89.0×3.0mm	1.0-2.0	376.0	430.9	207018.2
	102.0×3.0mm	1.0-2.0	399.7	462.3	202602.1
	114.9×3.0mm	1.0-2.0	425.7	468.5	204245.4
	127.3×3.0mm	1.0-2.0	398.3	469.8	201876.0
	139.2×3.0mm	1.0-2.0	362.0	464.3	202667.8
Series 2	165.1×3.0mm	1.0-2.0	477.3	545.2	205079.0
	220.0×5.0mm	1.0-2.0	494.3	557.4	204803.8
		2.5	503.7	569.4	205695.6
	270.0×4.8mm	1.0-2.0	358.7	447.8	205542.6
		2.5	355.7	460.5	205844.3

2. Specimen preparation, test set-up and procedure

All columns were supplied in 6m lengths, except for the 165.1×3 mm section, where only one 5 m length was supplied. In order to provide a range of columns from short to slender columns the supplied lengths were cut into 1 m, 1.5 m, 2.0 m and 2.5 m columns. After cutting the columns to the required lengths, the ends were milled flat, so that the load could be applied uniformly over the cross-sectional area.

The column strength is reduced by imperfections such as initial out of straightness and residual stresses. SANS 10162-1 [1] and EN 1993-1-1 [2] limits the initial out of straightness for most general shapes to L/1000. In this investigation the initial out of straightness was measured at mid-height of the column, using a thin metal wire and a pair of vernier calipers. In all the columns measured there was no visible initial out-of-straightness between the metal wire and the columns. It was concluded that the columns were straight and that initial out-of-straightness should not be considered. Residual stresses were also not considered since the manufacturing process of CHSs tends to distribute residual stresses uniformly, which results in greater compressive resistance [5].

The smaller sections (Series 1 specimens) were tested at the University of Johannesburg's Civil Engineering Science laboratory using a 500kN capacity Instron and the larger sections (Series 2 specimens) were tested at the University of the Witwatersrand using a 1500 kN Schenck Trebel compression machine. The (CHS) columns were compressed under concentric axial loads, between pin-ended conditions, at a loading rate of 5 mm/min. The slow loading rate allowed good observation of the failure modes during testing. In order to create pin-ended conditions, 2, 20 mm thick plates with spherical grooves and a steel ball were used at each end. Figure 5 shows typical experimental set ups for the two series of experiments.

(a) Instron (b) Schenck Trebel

Figure 1. Typical experimental set-up

3. Failure modes

All columns in Series 1, except 1m and 1.5m length columns, failed by overall flexural buckling (FB) followed by minor local buckling (LB) that occurred at mid-length of the columns (Figure 2(a)). Columns of larger outside diameter-to-thickness ratio (1 m and 1.5 m lengths) failed by interaction of local buckling and overall flexural buckling (L-FB) as shown in (Figure 2(b)).

Two sizes of columns (165.1x3.0 mm and 220.0x3.5 mm), tested with the Schenck Trebel machine, are classified as Class 4 or slender sections by both SANS 10162-1 [1] and EN 1993-1-1 [2]. As expected, these columns failed by local buckling (LB). Local buckling was manifested in these columns as an outward ring-type local buckle. Local ring-type buckling occurred at the bottom end of the 1m length column of the 165.1x3mm section (Figure 2(e)). For the 1.5 m and 2 m length columns, the local ring-type buckling accured at a third of the column height, from the top, and mid-height, respectively (Figure 2(c)). All the 220.0×3.5 mm columns failed by local ring-type buckling, at the mid-height of the 1 m, 2 m, and 2.5 m column lengths (Figure 2(c)). Although the 270.0×4.8 mm section was not expected to fail by buckling, all the 1.0 – 2.0 m columns failed by local ring-type buckling at the top end of the columns (Figure 2(d)), whilst the 2.5 m length column failed by flexural buckling (Figure 2(a)).

(a) (b) (c) (d) (e)

Figure 2. Typical failure modes

4. Local buckling

CHSs, subject to increasing compression load may experience local failure. This mode of failure is called local buckling. Local buckling is influenced by the outside diameter-to-thickness ratio (d/t) and the yield stress (f_y) of the sections, whilst overall flexural buckling or instability is influenced by the length of the column. To prevent local buckling from occurring, the diameter-to-thickness ratios of the elements must be limited. The maximum diameter-to-thickness ratios depend on the classification of the cross-section. Column cross-sections are classified into Class 1 (plastic), 2 (compact), 3 (semi-compact) and 4 (slender) in accordance with their behaviour in compression. In SANS 10162-1 [1], Classes 1, 2 and 3 have the same limits. The approach adopted in this standard is to separate sections that are stocky enough to be able to resist an axial stress equal to the yield stress without local buckling and those that buckle before the yield stress is reached. For easier of reference the three classes will be referred as Class 3. It should be noted that the steel sections cannot be designed on the basis of overall failure if the limiting values of a Class 3 section are exceeded. Sections with width-to-thickness ratios exceeding these values are referred to as Class 4 sections.

Table 2. Slenderness and diameter-to-thickness ratios

Series	CHS (dxt)	L (mm)	r (mm)	KL/r	d/t	f_y (MPa)	Limit (d/t)		Classification	
							SANS	EC3	SANS	EC3
Series 1	89.0x3.0	1000	30.4	32.9	29.7	375.2	61.3	31.3	Class 3	Class 1
		1500	30.4	49.3	29.7	375.2	61.3	31.3	Class 3	Class 1
		2000	30.4	65.7	29.7	375.2	61.3	31.3	Class 3	Class 1
	102.0x3.0	1000	35.0	28.6	34.0	392.5	58.6	41.9	Class 3	Class 2
		1500	35.0	42.8	34.0	392.5	58.6	41.9	Class 3	Class 2
		2000	35.0	57.1	34.0	392.5	58.6	41.9	Class 3	Class 2
	114.9x3.0	1000	39.6	25.3	38.3	416.2	55.3	39.5	Class 3	Class 2
		1500	39.6	37.9	38.3	416.2	55.3	39.5	Class 3	Class 2
		2000	39.6	50.5	38.3	416.2	55.3	39.5	Class 3	Class 2
	127.3x3.0	1000	44.0	22.7	42.4	395.9	58.1	53.4	Class 3	Class 3
		1500	44.0	34.1	42.4	395.9	58.1	53.4	Class 3	Class 3
		2000	44.0	45.5	42.4	395.9	58.1	53.4	Class 3	Class 3
	139.2x3.0	1000	48.2	20.8	46.4	362.1	63.5	58.4	Class 3	Class 3
		1500	48.2	31.1	46.4	362.1	63.5	58.4	Class 3	Class 3
		2000	48.2	41.5	46.4	362.1	63.5	58.4	Class 3	Class 3
Series 2	165.1x3.0	1000	57.3	17.4	55.0	484.7	47.5	43.6	Class 4	Class 4
		1500	57.3	26.2	55.0	484.7	47.5	43.6	Class 4	Class 4
		2000	57.3	34.9	55.0	484.7	47.5	43.6	Class 4	Class 4
	220.0x3.5	1000	76.6	13.1	62.9	494.3	46.5	42.8	Class 4	Class 4
		1500	76.6	19.6	62.9	494.3	46.5	42.8	Class 4	Class 4
		2000	76.6	26.1	62.9	494.3	46.5	42.8	Class 4	Class 4
		2500	76.6	32.7	62.9	503.7	45.7	41.0	Class 4	Class 4
	270.0x4.8	1000	93.8	10.7	56.3	354.4	64.9	59.7	Class 3	Class 3
		1500	93.8	16.0	56.3	354.4	64.9	59.7	Class 3	Class 3
		2500	93.8	26.7	56.3	348.6	66.0	60.7	Class 3	Class 3

For all the columns investigated (see Table 2), the outside diameter-to-thickness ratio (d/t) and the slenderness ratio (L/r) ranged from 29.7 to 46.4 and 20.8 to 82.2 for Series 1, respectively, and from 55.0 to 62.9 and 10.7 to 34.9 for Series 2, respectively. Parameters d, L and t, in Table 2, represent the outside diameter, clear length (which ranges from 1000 mm-2500 mm) and thickness of the CHSs. The slenderness range is significantly smaller than the compression slenderness limit of 200, as prescribed by SANS10162-1 [1].

For yielding to take place before local buckling, SANS10162-1 (SANS) requires that the maximum diameter-to-thickness ratio for CHSs be $d/t \leq 23000/f_y$. The maximum width-to-thickness ratios, prescribed by EN 1993-1-1, for Class 1, 2 and 3 are $d/t \leq 11750/f_y$, $d/t \leq 16450/f_y$ and $d/t \leq 21150/f_y$, respectively. A comparison of the diameter-to-thickness ratios of the sections tested and the code-predicted limits in Table 2 suggest that all columns, except the 165.1x3.0 and 220.0x3.5 CHSs, will yield before local buckling failure takes place.

5. Test results

A comparison of the test strength of the CHSs with the code-predicted strength, calculated using SANS 10162-1 (SANS) [1] and EN 1993-1-1 (EC3) [2] is shown in Table 3. In Table 3, N_{TEST} represent the experimental strength, N_{SANS} represent the strength predicted by SANS 10162-1 [1] and N_{EC3} represent the strength predicted by EN 1993-1-1 [2].

Table 3. Test and code-predicted strength of CHSs

Series	Size	d/t	KL/r	f_y (MPa)	N_{TEST} (kN)	N_{SANS} (kN)	N_{EC3} (kN)	N_{TEST}/N_{SANS}	N_{TEST}/N_{EC3}	Failure mode
Series 1	89.0x3.0	29.7	32.9	375.2	268.43	281.02	265.98	0.96	1.01	L-FB
		29.7	49.3	375.2	243.73	244.93	226.71	1.00	1.08	L-FB
		29.7	65.7	375.2	198.48	201.95	184.35	0.98	1.08	FB
	102.0x3.0	34.0	28.6	392.5	333.10	350.20	333.91	0.97	1.01	L-FB
		34.0	42.8	392.5	328.60	313.71	291.66	1.06	1.14	L-FB
		34.0	57.1	392.5	247.11	267.13	245.19	0.94	1.02	FB
	114.9x3.0	38.3	25.3	416.2	398.71	427.30	410.50	0.95	0.99	L-FB
		38.3	37.9	416.2	364.65	391.21	365.43	0.95	1.02	L-FB
		38.3	50.5	416.2	336.81	342.40	315.66	1.00	1.08	FB
	127.3x3.0	42.4	22.7	395.9	431.97	450.62	437.72	0.96	0.99	L-FB
		42.4	34.1	395.9	376.40	422.65	397.90	0.90	0.95	L-FB
		42.4	45.5	395.9	340.71	382.20	354.37	0.90	0.97	FB
	139.2x3.0	46.4	20.8	362.1	453.14	453.63	445.95	1.00	1.02	L-FB
		46.4	31.1	362.1	401.55	433.62	412.29	0.93	0.97	L-FB
		46.4	41.5	362.1	354.28	403.06	376.23	0.88	0.94	FB
Series 2	165.10x3.0	55.0	17.4	484.7	769.00	627.05	636.43	1.23	1.20	LB
		55.0	26.2	484.7	733.00	605.91	593.80	1.21	1.23	LB
		55.0	34.9	484.7	661.00	572.58	548.82	1.15	1.20	LB
	220.0x3.5	62.9	13.1	494.3	1061.00	864.96	978.02	1.23	1.10	LB
		62.9	19.6	494.3	1073.00	853.25	932.20	1.26	1.15	LB
		62.9	26.1	494.3	1040.00	833.59	885.90	1.25	1.17	LB
		62.9	32.7	503.7	997.00	805.95	845.12	1.24	1.18	LB
	270.0x4.8	56.3	10.7	354.4	1349.00	1428.81	1477.51	0.97	0.95	LB
		56.3	16.0	354.4	1362.00	1417.81	1425.20	0.99	0.98	LB
		56.3	26.7	348.6	1296.00	1360.75	1311.60	0.97	1.01	FB

To show the accuracy of SANS10162-1 [1] and EN 1993-1-1 [2], the experimental results of CHSs are compared with the code-predicted compressive resistance (Table 3). Generally EN 1993-1-1 [2] tends to model the tests results of all non-slender columns better than SANS10162-1 [1]. Evidence of this is provided by the test-to-predicted ratios of Class 1-3 sections, which are close 1 (Tables 2 and 3). In this group of sections, SANS10162-1 [1] models stockier sections or sections that experienced interaction of local and overall flexural buckling better than moderate slender sections or sections that experienced overall flexural buckling. Generally in non-slender sections (89.0x3.0 mm, 102.0x3.0 mm, 114.9x3.0 mm, 127.3x3.0 mm and 139.2x3.0 mm CHSs), SANS10162-1 [1] tends to overestimate the strength of the CHSs. The level of overestimation increases with increase in the slenderness ratio and diameter-to-thickness ratio. The most overestimated compressive resistance obtained is for the 139.2x3.0 mm columns, with a length of 2.0 m.

As shown in Table 3 and Figure 3, both standards do not model the strengths of slender sections well. SANS10162-1 [1] and EN 1993-1-1 [2] underestimate the strengths of the columns by an average of 22% and 18%, respectively. As expected all slender sections (165.10x3.0 mm and 220.0x3.5 mm) failed by local buckling. Local buckling was manifested as an outward ring-type local buckle. In general, EN 1993-1-1 [2] is more conservative than SANS10162-1 [1].

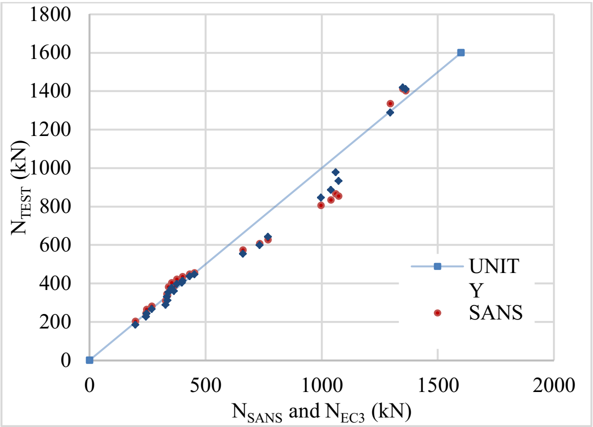

Figure 3. Comparison of experimental strength with the specified design strengths

6. Conclusions

All the Series 1 experiments specimens failed by flexural buckling accompanied by minor inward local buckling that occurred at the mid-length of the columns. Most Series 2 experiments specimens failed by local ring-type buckling and this was attributed to the large outside diameter-to-thickness ratio (d/t) of the specimens. Columns with large diameter-to-thickness ratio failed in a ductile manner (Figure 6-7) while those with small diameter-to-thickness ratio experienced a sudden decrease in capacity after reaching the maximum load. All slender sections (165.10x3.0 mm and 220.0x3.5 mm) failed by local buckling. Local buckling was revealed as an outward ring-type local buckle.

The test-to-predicted ratios of Class 1-3 sections of almost 1 show that EN 1993-1-1 [2] tends to model the tests results of all non-slender columns better than SANS10162-1 [1]. For the same group of sections, SANS10162-1 [1] models stockier sections or sections that experienced interaction of local and overall flexural buckling better than moderate slender sections or sections that experienced overall flexural buckling. Both standards do not model the strengths of slender sections well. As shown in Table 3, SANS10162-1 [1] and EN 1993-1-1 [2] underestimate the strengths of the columns by an average of 22% and 18%, respectively.

References

[1] SANS 10162-1, The structural use of steel - Part 1: Limit-state design of hot-rolled steelwork, South Africa National Standards, Pretoria, South Africa, 2011.
[2] EN 1993-1-1, Design of steel structures - Part 1.1: General rules and rules for buildings, British Standard Institution, 2005.
[3] CAN/CSA-S16-09, Limit states design of steel structures, Canadian Standards Association, Rexdale, Ontario, Canada, 2009.
[4] BS EN ISO 6892-1. Tensile testing of metallic materials: Part 1 - Method of test at room temperature. British Standard Institution; 2009.
[5] Kurobane Y, Packer JA, Wardenier J, Yeomans N. *Design guide for structural hollow section column connections*. CIDECT series no. 9. Cologne, Germany; 200