Advanced Steel Construction – Vol. 18 No. 3 (2022) 728–735
DOI:10.18057/IJASC.2022.18.3.10

EFFICIENCY OF DIFFERENT CONNECTIONS ON THE BEHAVIOUR OF COLD-FORMED SINGLE-ANGLE STEEL MEMBERS CONNECTED THROUGH ONE LEGUNDER AXIAL LOADING

R. Saleema begum [1, *] and P. Suresh Kumar [2]

[1] *Research Scholar, Department of Civil Engineering, University college of Engineering. Panruti, India*
[2] *Professor, Department of Civil Engineering, University college of Engineering. Panruti, India*
**(Corresponding author: E-mail: saleemabegum30@gmail.com)*

ABSTRACT

A detailed experimental program was performed using 36 cold-formed steel (CFS) single-angle column members attached by one leg was investigated subjected to axial compression loads. The key purpose of this research is to investigate the effect of slenderness ratio and different connection types on the load-carrying capacity of CFS angle sections under axial compression. The parameters investigated via the test program includes (a) angle sections with and without lipped profile, (b) sectional thicknesses (2 mm and 3mm), (c) slenderness ratios (λ = 20, 50, 80) from short to slender columns, and (d) type of connections i.e. two-bolt, three-bolt and welded connections. Results shown that the angle sections had a significant reduction in the load-carrying capacity when the slenderness ratio was increased from 20 to 80. Moreover, the mode of failure for short columns was changed from local buckling mode to combined local and flexural buckling for intermediate columns (λ = 50) and torsional-flexural buckling mode for long columns (λ = 80). Also, a detailed analytical study was carried out comparing the predictability of existing equations from different standards for angle sections under axial compression.

ARTICLE HISTORY

Received: 31 May 2022
Revised: 14 June 2022
Accepted: 19 June 2022

KEYWORDS

Cold-formed steel;
Bolted single-angle;
Welded single-angle;
Axial compression;
Local buckling;
Flexural-local buckling;
Flexural- torsional buckling

1. Introduction

The use of cold-formed steel (CFS) members especially in developing countries like India has significantly increased for the construction of residential, commercial and industrial buildings. Angle sections fabricated using CFS are predominantly used in different structural steel sections. Moreover, the fabrication of angle sections is relatively easier because of their simplified cross-section. Angle sections are usually connected to the other members through a single leg and are usually designed for predominant compression loads. It is worth mentioning that the effect of additional moments created due to eccentric connections and the shift of the effective centroid is neglected. The behaviour of axially loaded CFS angles sections with different end conditions has been previously investigated [1-7]. Popovic et al. [3]carried out the experimental investigations on CFSin-line galvanized equal angle sections. They concluded that theultimateload-carrying capacity of the stub column predicted by AS 4600 was found to conservative when compared to the experimental results. Young [9] performed tests on cold-formed steel plain angle columns with fixed end conditions. The test results were used as a benchmark for comparing the predictability of existing design equations suggested by American specifications and Australian/New Zealand standards for CFS sections. Ellobody and Young [11] developed a detailed finite element (FE) model for predicting thebehaviour of plain angle CFS columns. The developed model considered the effect of initial and geometric imperfections. The experimental results of 21 columns tested by Young [9] showed a good correlation with their FE model. Detailed experimental behaviour ofCFScolumns with unequal angles and non-symmetric lipped angle sections were also investigated [12-14]. Vishnuvardhan and Samuel knight [15] investigated thecompression behavior of CFS columns connected through single and compound plain angles. The load-carrying capacity and failure modes of CFS angles with different end conditions (i.e. ball, bolted and welded) were consideredas the test parameters to understand the overall behaviour of stub and short columns with single, double and starred angles. Zhou et al. [16] developed a non-linear FE model for predicting the ultimate compression strength of CFS angle sections connected through a single bolt. In the developed model, the contact between the angle section and gusset plate was also provided to achieve a conservative estimate of ultimate strength.

Several previous research papers have focused on the finite element modelling of CFS angle sections under axial loading which is used further for understanding the validity of existing equations from design standards [17-21]. Landesmann et al [20] carried outa detailed experimental behaviour to understand the slenderness effects (Short, intermediate and slender sections) of steel equal-leg angle columns with pin end conditions. The authors carried out the experimental investigation along with the nonlinear FEmodelling and analytical calculations (DSM equations). They concluded that the predictions from the validated FE model and improved analytical equations showed a close correlation with the test results of CFS columns with different slenderness limits. Silvestre et al. [22] documented a detailed design procedure for fixed and pin-ended columns with equal-leg angles. The height of the columns was varied in a range of short-to-intermediate level.The modified DSM approach showed a good ability to capture the ultimate strength of columns with short as well as intermediate lengths.Reviewing the existing literature, it is clear that only a handful of works have focused on the type of connections used for connecting the angle section to the column under axial compression. In specific, no studies have focussed on the effect of different connections (two bolted, three bolted and welded) and slenderness effects (short, intermediate and long) on the overall behavior ofCFS single-angle column members subjected to axial compression loading.The current work contributes towards filling the existing knowledge gap by presenting the experimental study on thebehaviour of 36single-angle CFS columns connected using two bolts, three bolts and welded connections for different column lengths (short, intermediate and long). Also, detailed analytical study was performed using the existing design equations to understand their predictability by comparison with the experimental results.

2. Research significance

From the critical review of existing studies, it is clear that only a few research works have focussed on understanding the axial behavior of single-angle CFS members. In specific, a knowledge gap can be witnessed to understand the effect of different connection types on the CFS columns subjected to axial compression loads. This study evaluates the effectiveness of different connection types on the compression behaviour of short, intermediate and slender CFS single-angle column members. The following are the specific objectives of the work:

(a) To quantify the effect of different connection types (two bolted, three bolted and welded) used for connecting the angle section to the column under axial compression.

(b) To understand the effect of slenderness ratio on the compression behaviour of CFS single- angle column members subjected to axial compression.

(c) To evaluate the validity of existing design equations for predicting the behaviour of CFS single-angle column members subjected to axial compression.

3. Experimental program

3.1. Details of test matrix

The test specimens were prepared from the cold-formed steel (CFS) material and connected using the gusset plates at both the ends, by bending, press braking and suitable welding procedures. Table 1 shows the overall details of the test program adopted in this work. The cold-formed sheets were supplied without holes and each specimen was tailor-made to the required lengths. Four different types of angels namely A 100 x 100 x 2, LA 82 x 82 x 20 x 2, A 100 x 100 x 3 and LA 83 x 83 x 20 x 3 were tested as a part of this work. Three different slenderness ratios (λ_v) were considered for each angle section such as 20, 50, and 80, where λ_v is the slenderness ratios about the minor principal axis and controlled the length of members. For the members connected using bolts, a single-angle section was bolted to a tee section at each end. The calculation length can bedefined as the design length which includes two times the end-plate thickness i.e., $L=L_0+2t+2L_g$ Where L is calculation length, L_0 is the net length between two end-plates; L_g is the unconnected length, tis the gusset plate thickness of T-plate. For the welded connection of CFS angle section, the experimental angles are CFS T-stubs which are arc-weldedusing two hot-rolled steel plates. The specimens tested as a part of this experimental program is shown in Fig. 1. Thisincludes a CFS column which is connected at the base by a single-angle with and without lipped profile.Two different thickness of CFS sections are used namely 2.0 mm and 3.0 mm. The specimen identification is denoted as P-CON-SL-t where, P- profile type (section with or without lippedprofile), CON – the type of connection (bolted or welded), SL –slenderness ratio (20 or 50 or 80) and t – thickness of CFS section. Example: In the nomenclature A-B2-20-2.0, A refers to the plain angle section without Lip; B2 refers to two bolted connection types; 20 refers to the value of slenderness ratio and 2.0 refers to the thickness of CFS section.

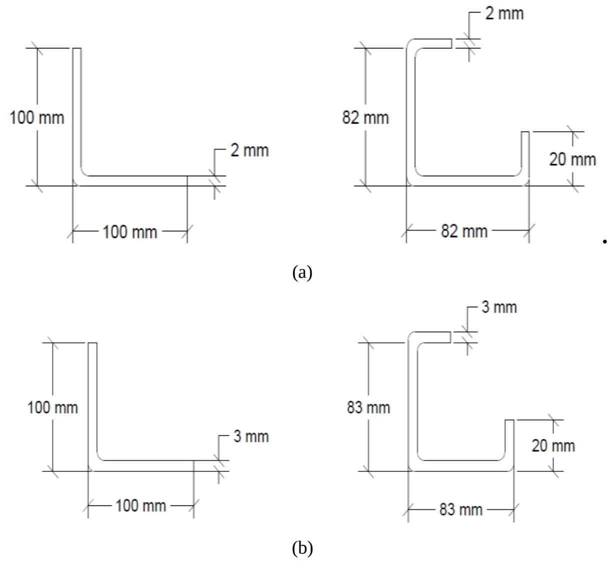


Fig. 1 Sectional details of plain and lipped section. (a) 2.0 mm thick and (b) 3.0 mm thick

3.2. Material characterisation

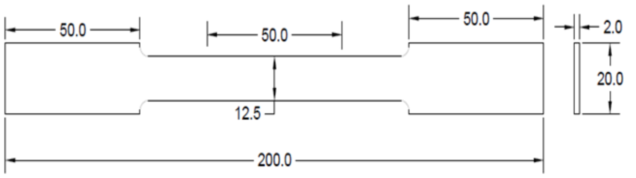
The coupon samples were prepared from the fabricated specimens as per the guidelines provided in ASTM 2013 [24]. The dimensions of the coupon samples are shown in Fig. 2(a). From each thickness of the test specimens (2.0 mm and 3.0 mm), six coupon samples were prepared and am average values were considering for reporting the mechanical properties of CFS sections. During sample preparation, the coupons were cut from the centre of the angle section as shown in Fig. 2(b). The stress-strain behaviour obtained for 2.0 mm and 3.0 mm CFS sections are shown in Fig. 2(c). Table 2 shows the mechanical properties of CFS section. For CFS sections with 2 mm thickness, the average yield strength and ultimate strain were found to be 268 MPa and 0.35 respectively. In the case of CFS sections with 3 mm thickness, the average yield strength and the ultimate strain were found to be 230 MPa and 0.44 respectively.

Table 1
Details of test specimen

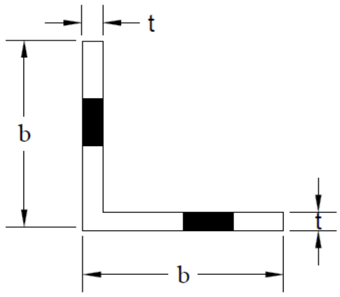
S. No	Specimen Size (mm)	Specimen ID	Area (mm^2)	Total length (mm)	Slenderness ratio (λ)	Connection Type
		A-B2-20-2.0		594		2Bolts
1	100 x 100x 2 (Plain Angle)	A-B3-20-2.0	396	644		3Bolts
		A-W-20-2.0		559	20	Weld
		LA-B2-20-2.0		594		2 Bolts
2	82 x 82 x 20 x 2 (Lipped Angle)	LA-B3-20-2.0	396	644		3 Bolts
		LA-W-20-2.0		559		Weld
		A-B2-50-2.0		1200		2 Bolts
3	100 x 100 x 2 (Plain Angle)	A-B3-50-2.0	396	1250		3 Bolts
		A-W-50-2.0		1165	50	Weld
		LA-B2-50-2.0		1200		2 Bolts
4	82x82x20x2 (Lipped Angle)	LA-B3-50-2.0	396	1250		3 Bolts
		LA-W-50-2.0		1165		Weld
		A-B2-80-2.0		1807		2 Bolts
5	100 x 100 x 2 (Plain Angle)	A-B3-80-2.0	396	1857		3 Bolts
		A-W-80-2.0		1772	80	Weld
		LA-B2-80-2.0		1807		2 Bolts
6	82 x 82 x 20x2 (Lipped Angle)	LA-B3-80-2.0	396	1857		3 Bolts
		LA-W-80-2.0		1772		Weld
		A-B2-20-3.0		592		2 Bolts
7	100 x 100 x 3 (Plain Angle)	A-B3-20-3.0	591	642		3 Bolts
		A-W-20-3.0		557	20	Weld
		LA-B2-20-3.0		592		2 Bolts
8	83x83x20x3 (Lipped Angle)	LA-B3-20-3.0	591	642		3 Bolts
		LA-W-20-3.0		557		Weld
		A-B2-50-3.0		1195		2 Bolts
9	100 x 100 x 3 (Plain Angle)	A-B3-50-3.0	591	1245		3 Bolts
		A-W-50-3.0		1160	50	Weld
		LA-B2-50-3.0		1195		2 Bolts
10	83x83x20x3 (Lipped Angle)	LA-B3-50-3.0	591	1245		3 Bolts
		LA-W-50-3.0		1160		Weld
		A-B2-80-3.0		1799		2 Bolts
11	100 x 100 x 3 (Plain Angle)	A-B3-80-3.0	591	1849		3 Bolts
		A-W-80-3.0		1764	80	Weld
		LA-B2-80-3.0		1799		2 Bolts
12	83 x 83 x 20x3 (Lipped Angle)	LA-B3-80-3.0	591	1849		3 Bolts
		LA-W-80-3.0		1764		Weld

Table 2
Mechanical Properties of CFS

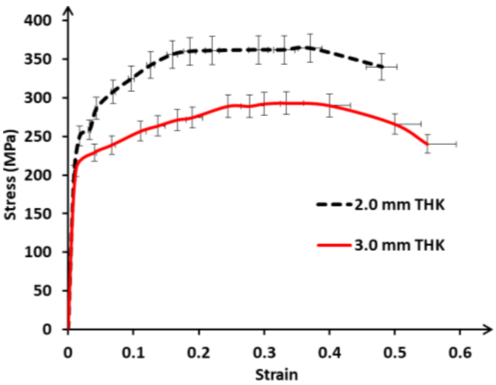
S. No	Thickness of CFS	Elastic Modulus	Yield Strength (fy)	Ultimate Strength (fu)	fu/fy	Elongation
1.	2.0 mm	200 GPa	268.0 MPa	362.0 MPa	1.35	35.85%
2.	3.0 mm	200 GPa	229.8 MPa	306.1 MPa	1.33	44.24%



(a)



(b)



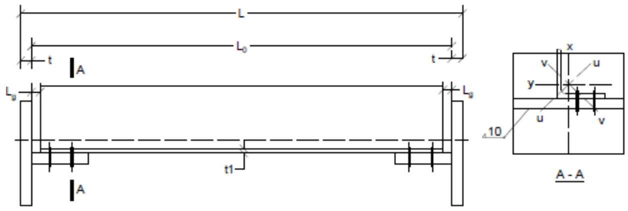
(c)

Fig. 2 Coupon details and stress – strain behaviour of CFS with different thickness

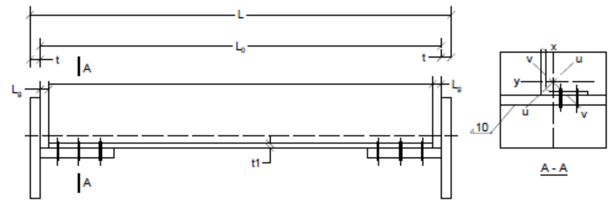
3.3. Test setup and instrumentation details

The specimens are tested using theuniversal testing machine (UTM) of 1000 kN capacity. In total, thirty-six single angle column members connected to the tee section were tested under axial compression to determine their ultimate strength and corresponding failure modes. The web of the tee sections is connectedusing either bolted or welded type as shown in Fig. 3. Three different slenderness ratios are evaluated by testing short, intermediate and long columns.A 10 mm thick end plates of size 100 x 100 mm and 150 x 100 mm are welded and made as T-stub gusset plates attached to the specimens at each end for bolted and welded connection. The specimens were fixed vertically by gripping the gusset plates and were tested to failure. For each test, the load was increased at a faster rate in the elastic range (5 kN/sec) and a slower range in the plastic range till the specimen failed. To understand the post-buckling behaviour of test specimens, the measurements were also done beyond the peak load. The procedure is repeated till the failure stage is reached for all the specimens.

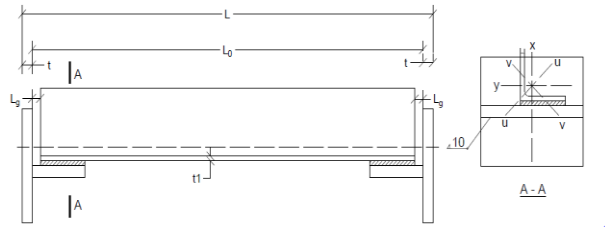
The details of the test setup and instrumentations used during the application of compression loads are shown in Fig. 4. Carehas been taken to load the specimen vertically using the gusset plate. Dial gauges of least count 0.01 mm were used to measure the axial shortening of the member and lateral deflections. Dial gauges were placed at the mid-height of the angle section and at one-fourth of the height of the angle section with their end touching the web and flange of the specimens for measuring the lateral deflections. To measure the axial shortening of the test specimen,two dial gauge was placed with its end touching the movable head of the column testing machine. Electrical resistance strain gauges were used to measure the strains at mid-height of the angle section. A strain indicator with 5 channels was used to record the strain measurements. Strain gauge and dial gauge readings were measured at every increment of load and the load was increased until the specimen attains failure.



(a)

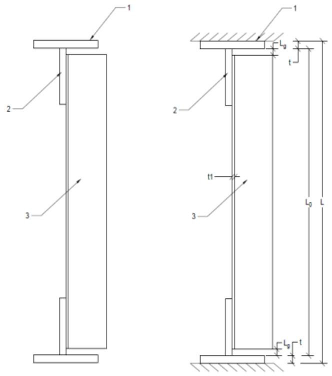


(b)



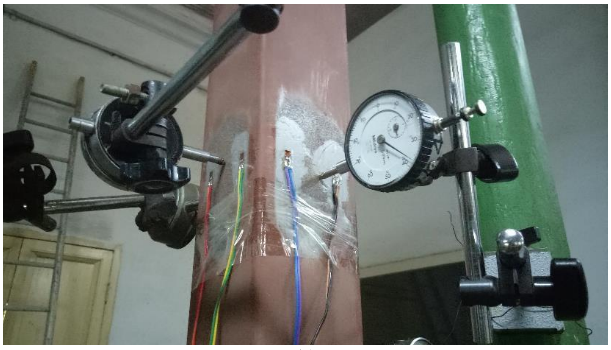
(c)

Fig. 3 Schematic representation of different end connections used.



(a)

(b)



(c)

Fig. 4 Test setup and instrumentation details

4. Results and discussion

The test results obtained for single –angle column members with different slenderness ratio, sectional thickness and type of connections are discussed in the following section to understand the axial capacity and the change in failure mode.

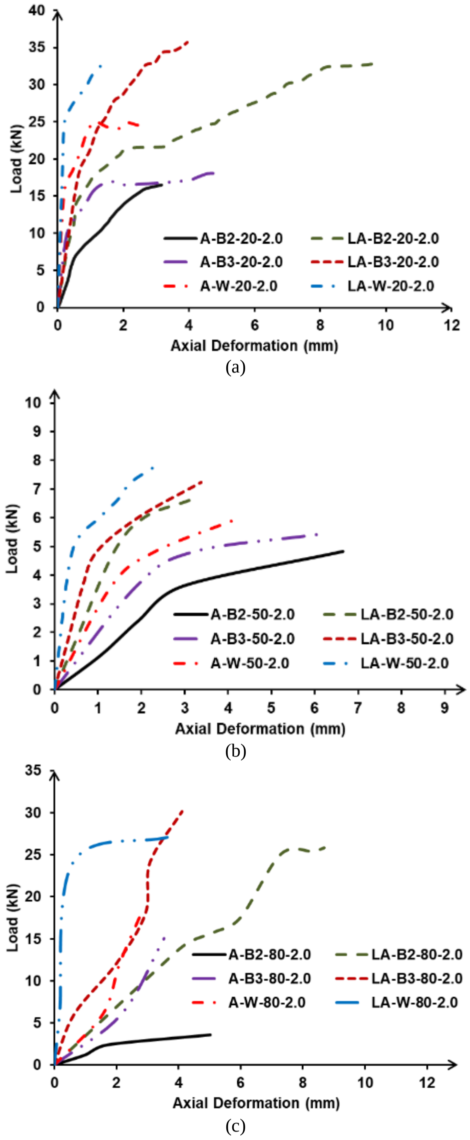


Fig. 5 Overall behavior of specimens with 2.0 mm sectional thickness

4.1. Single-angle column members with 2.0 mm thickness

Table 3 highlights the peak load for different specimens and their corresponding failure modes. The load-displacement behaviour obtained for single angle columns with different connections is shown in Fig. 5. It is clear that the axial load carrying capacity of specimens reduced significantly with the increase in slenderness ratio. Single-angle column members with a slenderness ratio of 20 and connected using 2 bolts had an axial load of 16.50 kN. With the increase in slenderness ratio, the axial capacity of columns reduced significantly by 70.7% and 85.3% compared to A-B2-20-2.0. However, for the same specimen (A-B2-20-2.0), the axial load capacity increased by 9.4% and 50.6% when the column and single angle is connected using 3 bolts and welded connection, respectively. Similarly, for the specimen A-B3-20-2.0, the peak load was found to be 18.05 kN. When the slenderness ratio is increased to 50 and 80, the peak load reduced by 69.9% and 83.5% respectively compared to A-B3-20-2.0. For the specimens connected through the welded section (A-W-20-2.0), the axial load capacity was found to be 24.86 kN which is more than 50% for the similar specimen with 2 bolt connection (A-B2-20-2.0). When the slenderness ratio was increased to 50 and 80, the axial load capacity reduces by 73.4% and 80.7% respectively compared to the specimen A-W-20-2.0.

Use of a lipped profile instead of plain angles helped in increasing the axial capacity of members with different connections. Comparison for the specimen with a slenderness ratio of 20 and 2 bolted connections, the peak strength of the lipped section increased by 98.8% when compared to the specimen without lip. Similar results were observed for lipped profile when the slenderness ratio is increased more than 20. With the increase in slenderness ratio (LA-B2-50-2.0 and LA-B2-80-2.0), the lipped angle sections were more effective under axial loading and helped in enhancing the overall load capacity by 24.82% and 106.6% when compared to similar specimens without lip (A-B2-50-2.0 and A-B2-80-2.0). For the specimen connected using 3 bolts (LA-B3-20-2.0), the peak load was found to be 35.67 which is more than 97.6% compared to a similar specimen without lipped angle. When the slenderness ratio is increased, the lipped angle section also shows a significant reduction in axial resistance due to the secondary effects and the reduction was found to be 79.8% and 83.1% respectively. For lipped angles connected by welding, the peak load increased by 58.6% compared toasimilar specimen without lipped profile. Moreover, the specimens LA-W-80-2.0 and LA-W-80-2.0 had axial load reduction of about 80.2% and 84.2% respectively when compared to the specimen with low slenderness ratio (LA-W-20-2.0).

Table 3
Test results for specimens with 2.0 mm thickness

Specimen ID	Angle size (mm)	b/t ratio	Slenderness ratio (λ)	Peak load (PEXP) kN	Failure mode
A-B2-20-2.0	100 x 100 x 2.0	50	20	16.50	L
A-B2-50-2.0			50	4.82	L+F
A-B2-80-2.0			80	2.42	F+T
A-B3-20-2.0	100 x 100 x 2.0	50	20	18.05	L
A-B3-50-2.0			50	5.42	L+F
A-B3-80-2.0			80	3.01	F+T
A-W-20-2.0	100 x 100 x 2.0	50	20	24.86	L
A-W-50-2.0			50	6.62	L+F
A-W-80-2.0			80	4.81	F+T
LA-B2-20-2.0	82 x 82 x 20 x2.0	41	20	32.80	L
LA-B2-50-2.0			50	6.02	L+F
LA-B2-80-2.0			80	5.00	F+T
LA-B3-20-2.0	82 x 82 x 20 x 2.0	41	20	35.67	L
LA-B3-50-2.0			50	7.22	L+F
LA-B3-80-2.0			80	6.02	F+T
LA-W-20-2.0	82 x 82 x 20 x 2.0	41	20	39.45	L
LA-W-50-2.0			50	7.83	L+F
LA-W-80-2.0			80	6.62	F+T

Note: L – Local Buckling; L+F – Combination of local and flexural buckling and T – Flexural- torsional buckling

4.2. Single-angle column members with 3.0 mm thickness

Table 4 highlights the peak load for specimens with 3.0 mm thickness. Moreover, the overall load-displacement behaviour obtained for single angle columns with different connections is shown in Fig. 6. In this section, the comparison is made highlighting the effect of different slenderness ratios and types of connections. For the control specimen with a slenderness ratio of 20 and connected by two bolts (A-B2-20-3.0), the peak load was found to be 34.87 kN.Increase in the slenderness ratio by 50 and 80, the axial capacity reduced by 77.5% and 82.7% respectively. For the specimen with 3 bolted connections and slenderness ratio 20 (A-B3-20-3.0), the axial load increased marginally by 6.1% when compared to 2 bolted sections (A-B2-20-3.0).When the slenderness ratio is increased by 50 and 80, the axial strength reduced by 77.2% and 80.5% respectively. The use of a welded connection was more effective compared to the other two connections used. For specimen A-W-20-3.0, the peak compressive strength increased by 21.7% when compared to the specimen with two bolted connections (A-B2-20-3.0). For similar specimen, the peak strength reduced by 75.9% and 80.1% for specimens with slenderness ratio of 50 and 80 respectively.

For the specimens with 3.0 mm thickness, the use of a lipped section profile helps in enhancing the peak compressive strength. Comparing the behaviour of single angled column members with lipped profile, the peak strength increased by 51.3%, 70.4% and 54.1% respectively when compared to the similar specimens A-B2-20-3.0, A-B3-20-3.0, A-W-20-3.0 respectively without a lipped profile. Considering the effect of different connections, the lipped angle sections connected using 3 bolts showed better performance when compared to the 2 bolt and welded connection. The effect of slenderness ratio also played a significant role in the compressive strength of lipped angle column sections. For the 2 bolt connection, the peak compressive strength reduced by 81.7% and 82.9% for specimens with slenderness ratio 50 (LA-B2-50-3.0) and 80 (LA-B2-80-3.0) respectively.Similarly, for the welded specimens with lipped angles, the peak load reduced by 82.5% and 84.4% for specimens with slenderness ratio 50 (LA-W-50-3.0) and 80 (LA-W-80-3.0) respectively

Table 4
Test results for specimens with 3.0 mm thickness

Specimen ID	Angle size (mm)	b/t ratio	Slenderness ratio (λ)	Peak load (PEXP) kN	Failure mode
A-B2-20-3.0	100 x 100 x 3.0	33.33	20	34.87	L
A-B2-50-3.0			50	7.83	L+F
A-B2-80-3.0			80	6.02	F+T
A-B3-20-3.0	100 x 100 x 3.0	33.33	20	37.00	L
A-B3-50-3.0			50	8.43	L+F
A-B3-80-3.0			80	7.23	F+T
A-W-20-3.0	100 x 100 x 3.0	33.33	20	42.43	L
A-W-50-3.0			50	10.24	L+F
A-W-80-3.0			80	8.43	F+T
LA-B2-20-3.0	82 x 82 x 20 x3.0	27.66	20	52.75	L
LA-B2-50-3.0			50	9.63	L+F
LA-B2-80-3.0			80	9.03	F+T
LA-B3-20-3.0	82 x 82 x 20 x 3.0	27.66	20	63.03	L
LA-B3-50-3.0			50	10.84	L+F
LA-B3-80-3.0			80	9.63	F+T
LA-W-20-3.0	82 x 82 x 20 x 3.0	27.66	20	65.38	F
LA-W-50-3.0			50	11.45	L+F
LA-W-80-3.0			80	10.24	F+T

Note: L – Local Buckling; F – Flexural Buckling; L+F – Combination of local and flexural buckling and T – Flexural torsional buckling

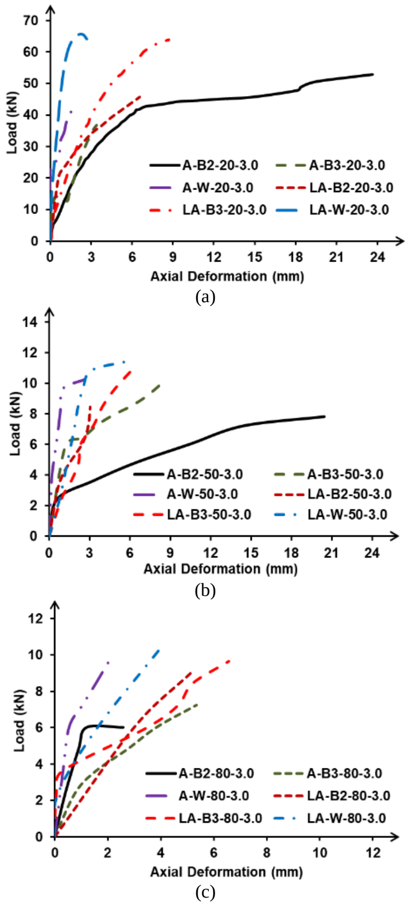


Fig. 6 Overall behavior of specimens with 3.0 mm sectional thickness

4.3. Load – strain behaviour

The load - strain behaviour of single-angle column members with different slenderness ratio and connections are shown in Figs. 7 and 8. The negative values indicate the compressive strain and the positive value indicates the tensile strain. For the specimens with a slenderness ratio of 20, the welded section showed better performance interns of excessive compressive and tensilestrain values. The compressive strength showed a value close to 1400 μm/m exhibiting a large ductile response before failure. Also, the stiffness of the specimen improves with the change in connection type. The single-angle column members with welded connection showed a higher value of initial stiffness followed by the members with 3 bolt connection and then the 2 bolt connection. With the increase in slenderness ratio, the compressive strain reduced significantly. However, the tensile strain values showed some increase due to the increased secondary effect i.e., large lateral deflections before failure. For the single-angle column members without a lipped profile and having a slenderness ratio of 80, the compressive strain was close to 1600 μm/m showing good axial resistance irrespective of the large lateral deflections.

Similar behavior was observed for the sections with 3.0 mm thickness except for the fact that most of them were subjected to predominate compressive strain. This behaviour also indicates an increase in overall effectiveness with the increase in sectional thickness. In the case of sections with 3 mm thickness, the welded connection exhibited a better performance in resisting excessive compressive strains. For the slenderness ratio of 20, the maximum compressive strain resistance of 1400 μm/m is exhibited by the specimen A-W-20-3.0.Similarly, a compressive strain value of 1600 μm/m was attained by specimen A-W-80-3.0 showing their ability to take excessive axial strain irrespective of the large secondary effects (lateral deformation) due to the slenderness ratio of 80. Provision of lipped profile did not significantly enhance the performance in terms of compressive strain resistance. Nevertheless, the peak compressive strength increased when compared to the specimens without a lipped profile.

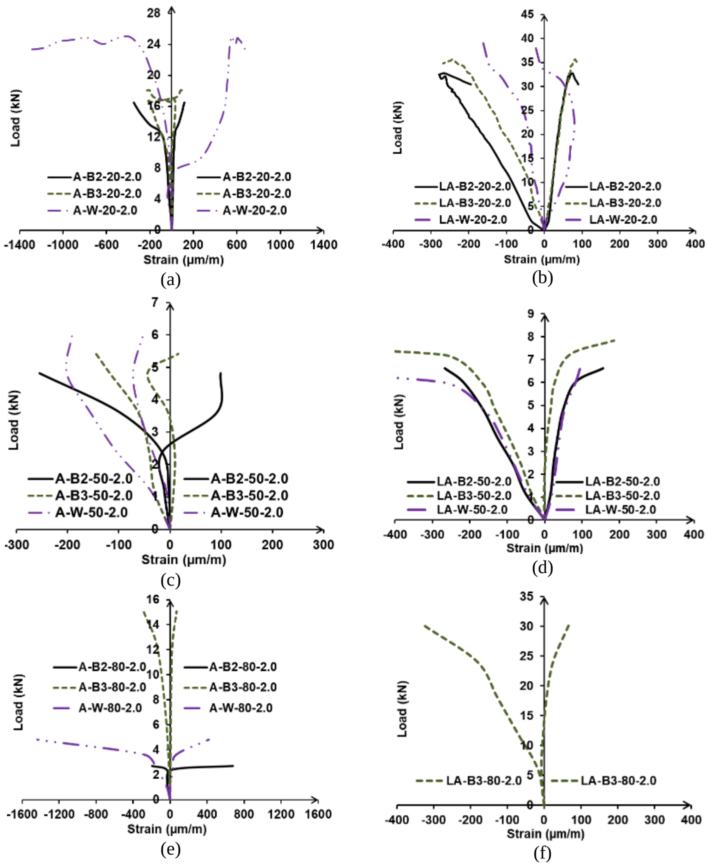
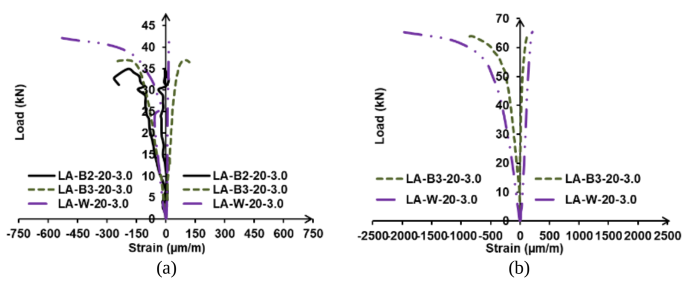


Fig. 7 Overall Load – Strain Behavior with 2.0 mm sectional thickness



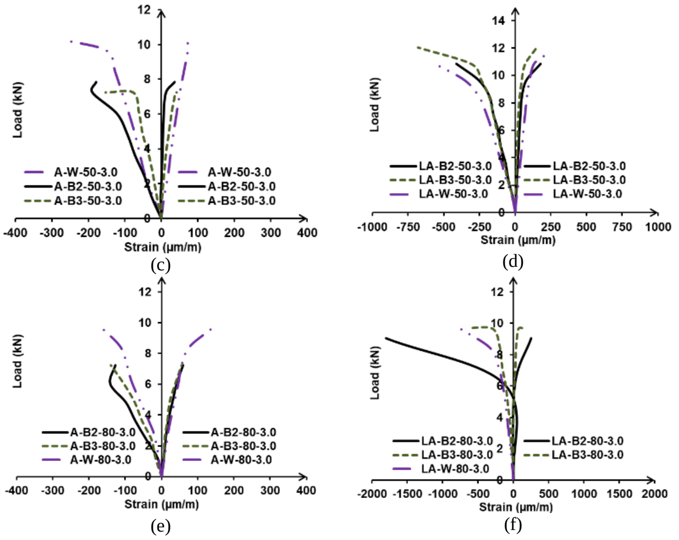


Fig. 8 Overall load – strain behavior with 3.0 mm sectional thickness

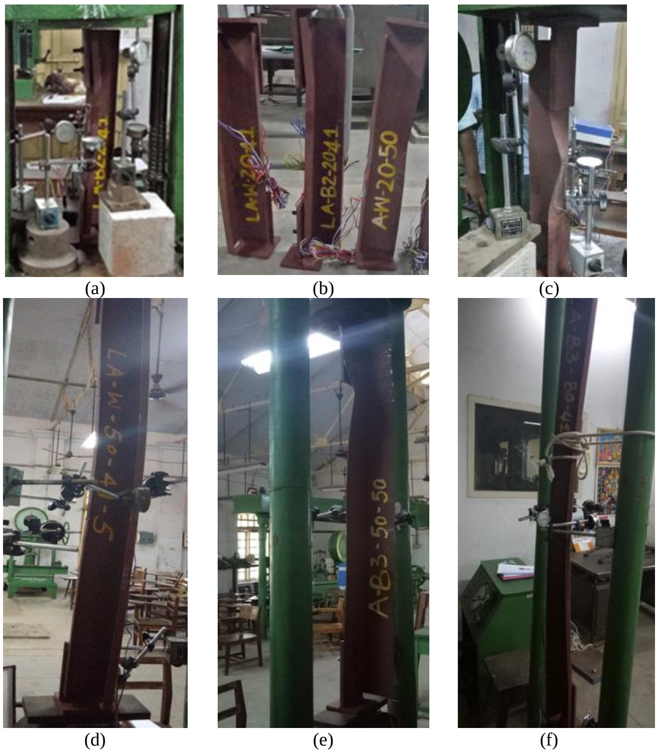


Fig. 9 Failure of specimens with different slenderness ratio and connections

4.4. Failure mode comparison

The failure mode of single-angle column sections with different thickness and slenderness ratio is shown in Fig. 9. All the angle sections with and without lipped profile and connected using two bolts had failure due to local buckling. The occurrence of local buckling failure is characterised by the presence of flexural deformation of the plate initiated either at the one-third or mid-height of the column. With the increase in number of bolts used for connecting the column and the single-angle, the occurrence of local buckling alone is prevented and the failure is due to a combination of flexural and local buckling. No single-angle column members had failure due to flexural buckling only i.e., the rigid body movement or global movement of the entire column members. For all the members connected using 3 bolts, the local-flexure buckling failure mode occurred which is characterised by the occurrence of global movement of the entire member along with the localised plate deformation either at the mid-height or one-third height of the section. The interaction between the local and flexural buckling relies largely on the slenderness ratio of the section and the dominance of flexural buckling can be witnessed significantly with the increase in slenderness ratio. For the single-angle welded column section, the failure mode is due to flexural-torsional buckling. This failure mode involves a combination of member bending and twisting as well as any local buckling of slender elements. Due to the low-torsional rigidity of thin walled members, the compression flange tends to buckle in the inward direction. The failure mode type of flexural torsional buckling is different from the lateral torsional buckling which involve a twisting of the entire cross section about its shearcentre. When the connection type for single-angle column sections are changed from two or three bolted connection, the failure mode is converted from local buckling or flexural-local buckling into flexural-torsional buckling irrespective of the slenderness ratio and sectional thickness.

5. Prediction analysis of existing design standards

5.1. IS 801: 2005 [26]

Singly symmetric shapes or intermittently fastened singly- symmetric components of built-up shapes Having Q =1.0 which may be subject to Torsional Flexural Buckling-singly symmetric shapes subject to both axis compression and bending applied in the plane of symmetry shall be proportioned to meet the following four requirements as applicable.

$$\sigma_{ex} = \pi^2E/(KL/r_x)^2 \quad (1)$$

For $F_a/F_{a1} + f_{b1}C_m/F_{b1}(1-F_a/F') \leq 1$

$$F_a/F_{ao} + f_{b1}/F_{b1} \leq 1 \quad (2)$$

For $f_a/F_{a1} \leq 0.15$

$$F_a/F_{a1} + f_{b1}/F_{b1} \leq 1.0 \quad (3)$$

If the point of application of the eccentric load is located on the side of the centroid opposite from that of the shear centre, that is if e is positive, then the average compression stress fa shall not also not exceed Fa given below

For $\sigma_{TF} \geq 0.5F_y$

$$Fa = 0.522\, F_y - F_y^2/7.67\sigma_{TF} \quad (4)$$

For $\sigma_{TF} < 0.5F_y$

$$Fa = 0.522\, \sigma_{TF} \quad (5)$$

Where σ_{TF} shallbe determined according to the formula:

$$\sigma_{TF}/\sigma_{TFo} + C_{TF}\sigma_{b1}/\sigma_{bt}(1-\sigma_{TF}/\sigma_e) = 1.0 \quad (6)$$

From the mean compressive stress values, the peak strength of the CFS columnscan be estimated by multiplying the effective area value and the calculatedaverage allowablestress from the above mentioned equation.

5.2. American iron and steel institute AISI 2016

The design rules of the current AISI design code is based on the research work by Popovic et al. [1].The nominal axial strength (P_n) is calculated using equation (9).

$$P_n = A_eF_n \quad (9)$$

The ultimate design strengthcan be estimated using equation (10)

$$P_u = P_n/1.80 \quad (10)$$

Considering the Allowable Stress Design (ASD), the peak load of the CFS column members can be estimated as given in equation (11)

$$P_u = 0.85 \times Pn \quad (11)$$

$F_n$is determined as per the below equations (12) or (13)

For $\boldsymbol{\lambda}_c \leq 1.5$

$$F_n = [0.658\boldsymbol{\lambda}_c^2]\, F_y \quad (12)$$

For $\boldsymbol{\lambda}_c > 1.5$

$$F_n = [0.877/\boldsymbol{\lambda}_c^2]F_y \quad (13)$$

Where

$\boldsymbol{\lambda}_c = \sqrt{F_y/Fe}$

WhereFe= least of the elastic flexural, torsional and torsional- flexural buckling stresses determined appropriately.For the sections subjected to flexural-torsional buckling or torsional buckling, the elastic flexural buckling stress can be calculated using the following formula

$$F_{cre} = \frac{\pi^2 E}{(KL/r)^2} \tag{14}$$

E = Modulus of elasticity of steel
K = Effective length factor
L = laterally unbraced length of member
R = Radiusofgyrationoffullunreducedcross-sectionaboutaxisofbuckling

The load capacity in combined axial and bending is determined using the following equation.

$$\frac{\bar{P}}{P_a} + \frac{\overline{M_x}}{M_{ax}} + \frac{\overline{M_y}}{M_{ay}} \leq 1.0 \tag{15}$$

5.3. British standard BS:5950 (Part 5)-2002

For CFS column sections with at least one axis of symmetry and subjected to torsional – flexural buckling mode, the peak compressive strengthcan be estimatedusing the provisions ofBS:5950 (Part 5)-2002 [27]. The stress corresponding to the torsional–flexural buckling can be calculated using equation (15) where the effective length (LE)is substituted by a factored effective length α are calculatedusing the following conditions below.

$$F_C/P_{CS} + M_x/M_{cx} + M_y/M_{CY} \leq 1 \tag{16}$$

For beams not subject to lateral buckling the following relationship should be satisfied.

$$F_C/P_C + M_x/C_{bx}M_{cx}(1\text{-}F_C/P_{EX}) + M_Y/C_{by}M_{cy}(1\text{-}F_C/P_{EY}) \leq 1 \tag{17}$$

For beams subject to lateral buckling the following relationship should be satisfied.

$$F_C/P_C + M_x/ M_b + M_y /C_{by}M_{cy}(1\text{-}F_c/P_{cy}) \leq 1 \tag{18}$$

The magnitudes of moments M_x and M_y should take in to account any moment induced by the change in neutral axis position of the effective cross-section caused by the axial load. In the determination of C_{bx} and C_{by} the effects of change in the neutral axis position of the effective cross section caused by the axial load. In the determination of $C_{bx}$and C_{by} the effects of change in the neutral axis position on the moment variation may be neglected.

5.4. Comparison of predictions with the test results

Table 5 and Table 6 highlights the comparison of test results with the peak strength predictions obtained from the analytical calculations for different sectional thickness. It can be witnessed that the results obtained from the analytical calculations under-predicted the tests. The variation of the ultimate strength ratio of experiments and analytical studies (P_{EXP} / P_{Code}) for single angle column members with 2.0 mm sectional thickness was found to be 1.07, 1.40 and 3.92 for IS 801: 2005, AISI 2016 and BS 5950: 2002 respectively. Similarly, the variation of the ultimate compression strength ratio of tests and analytical predictions (P_{EXP} / P_{Code}) for single angle column members with 3.0 mm sectional thickness was found to be 0.63, 1.85 and 13.90 for IS 801: 2005, AISI 2016 and BS 5950: 2002 respectively. From the above prediction range, it is very clear that the values obtained from IS 801-2005 were conservative and close when compared to the test results of specimens with 2.0 mm sectional thickness. However, when the sectional thickness is increased to 3.0 mm, the analytical predictions over-predicted the experimental results by more than 35%. For both the sectional thickness, the BS code and AISI code were found to provide a conservative estimate of compression load. It is worth mentioning that the analytical predictions obtained from the British standards were too-much conservative. The prediction range increases (COV = 13.92) drastically when the sectional thickness is increased to 3.0 mm. The predictions from AISI 2016showed a close and conservative estimate of test results.

Table 5
Test results for specimens with 2.0 mm thickness

Specimen ID	Test (PEXP) kN	IS 801: 2005 (PIS) kN	AISI 2016 (PAISI) kN	BS 5950:2002 (PBS) kN	PIS/PEXP	PAISI/PEXP	PBS/PEXP
A-B2-20-2.0	16.5	4.23	4.15	4.15	0.26	0.25	0.25
A-B2-50-2.0	4.82	3.47	4.10	1.35	0.72	0.85	0.28
A-B2-80-2.0	2.42	3.45	4.01	0.58	1.43	1.66	0.24
A-B3-20-2.0	18.05	4.23	4.16	5.97	0.23	0.23	0.33
A-B3-50-2.0	5.42	3.48	4.13	2.27	0.64	0.76	0.42
A-B3-80-2.0	3.01	3.47	4.07	0.99	1.15	1.35	0.33
A-W-20-2.0	24.86	4.23	4.16	9.63	0.17	0.17	0.39
A-W-50-2.0	6.62	3.48	4.14	3.55	0.53	0.63	0.54
A-W-80-2.0	4.81	3.47	4.11	1.22	0.72	0.85	0.25
LA-B2-20-2.0	32.8	10.08	17.85	4.16	0.31	0.54	0.13
LA-B2-50-2.0	6.02	5.24	4.92	1.35	0.87	0.82	0.22
LA-B2-80-2.0	5	3.45	2.85	0.59	0.69	0.57	0.12
LA-B3-20-2.0	35.67	10.91	19.85	5.97	0.31	0.56	0.17
LA-B3-50-2.0	7.22	7.17	6.24	2.27	0.99	0.86	0.31
LA-B3-80-2.0	6.02	4.42	5.25	0.99	0.73	0.87	0.17
LA-W-20-2.0	39.45	11.69	21.85	8.82	0.30	0.55	0.22
LA-W-50-2.0	7.83	6.59	5.49	3.56	0.84	0.70	0.45
LA-W-80-2.0	6.62	5.51	4.25	1.48	0.83	0.64	0.22
Mean COV					0.65	0.71	0.28

Table 6
Test results for specimens with 3.0 mm thickness

Specimen ID	Test (PEXP) kN	IS 801: 2005 (PIS) kN	AISI 2016 (PAISI) kN	BS 5950:2002 (PBS) kN	PIS/PEXP	PAISI/PEXP	PBS/PEXP
A-B2-20-3.0	34.87	14.22	6.77	4.99	0.41	0.19	0.14
A-B2-50-3.0	7.83	3.15	6.73	1.54	0.40	0.86	0.20
A-B2-80-3.0	6.02	2.99	6.67	0.67	0.50	1.11	0.11
A-B3-20-3.0	37	14.22	6.77	7.25	0.38	0.18	0.20
A-B3-50-3.0	8.43	3.19	6.75	2.57	0.38	0.80	0.31
A-B3-80-3.0	7.23	3.11	6.72	1.13	0.43	0.93	0.16
A-W-20-3.0	42.43	14.22	6.77	11.91	0.34	0.16	0.28
A-W-50-3.0	10.24	3.22	6.76	4.02	0.31	0.66	0.39
A-W-80-3.0	8.43	3.16	6.74	1.68	0.37	0.80	0.20
LA-B2-20-3.0	52.75	47.21	35.59	4.30	0.89	0.67	0.08
LA-B2-50-3.0	9.63	3.38	2.79	1.33	0.35	0.29	0.14
LA-B2-80-3.0	9.03	2.63	2.34	0.58	0.29	0.26	0.06
LA-B3-20-3.0	63.03	50.20	37.39	6.25	0.80	0.59	0.10
LA-B3-50-3.0	10.84	3.88	3.07	2.22	0.36	0.28	0.20
LA-B3-80-3.0	9.63	3.10	2.62	0.98	0.32	0.27	0.10
LA-W-20-3.0	65.38	53.28	39.26	10.27	0.81	0.60	0.16
LA-W-50-3.0	11.45	4.29	3.31	3.47	0.37	0.29	0.30
LA-W-80-3.0	10.24	3.46	2.83	1.43	0.34	0.28	0.14
Mean COV					0.45	0.51	0.18

6. Summary and conclusions

The following major conclusions can be drawn from the limited results presented in this work:

- Use of two bolt connection showed a negative effect with the increase in slenderness ratio i.e., with the increase in slenderness ratio from 20 to 80, the peak compression load reduced significantly by more than 80%. All the single-angle column members connected with 2 bolts had failure due to local buckling mode.
- In the case of single-angle column members with three bolted connections, the peak strength increased in a range of 10% when compared to the specimens with two bolted connections. Moreover, the failure mode converted from local buckling mode (2 bolted connection) to flexural-local buckling mode (3 bolted connection).
- Use of welded connection was found to most efficient among the three investigated as a part of this study. The welded connection used for connecting the angle section to the column section helped in significantly enhancing the peak strength and strain. Moreover, the specimens were found to fail under flexural-torsional buckling mode.
- Increase in the value of slenderness ratio showed a considerable reduction in the ultimate compressive strength of single-angle column members. However, the slenderness ratio didn't have any effect of the failure mode of members which were more dependent on the type of connections used.
- The analytical procedure used in this study showed a good predictability for the ultimate compressive strength of single-angle column members with and without lipped profile. Only, the predictions obtained from the AISI code were conservative and close to the test results.

References

[1] Madugula MKS, Prabhu TS, Temple MC. Ultimate strength of concentrically loaded cold-formed angles. Canadian J. Civ. Eng. 1983; 10(1): 60-68.
[2] Madugula MKS, Ray SK. Ultimate strength of eccentrically loaded cold -formed angles. Canadian J. Civ. Eng. 1984; 10(1): 225-233.
[3] Popovic D, Hancock HJ, Rasmussen KJR. Axial compression tests of cold formed angles. J. Struct. Eng. 1999; 125(5): 515-52
[4] Dhanalakshmi M, Shanmugam, NE. Stub column tests on cold-formed steel angle sections. International Specialty Conference on Cold-Formed Steel Structures, Missouri U.S.A, October 19th - 20th, 2000.
[5] Popovic D, Hancock HJ, Rasmussen KJR. Compression tests on cold formed angles loaded parallel with a leg. J. Struct. Eng. 2001; 127(6):600-607.
[6] Dubina D, Ungureanu V. Effect of imperfections on numerical simulation of instability Behaviour of Cold-formed steel members. Thin Walled Struct. 2002; 40(3): 239-262.
[7] Narayanan S, Mahendran M. Ultimate capacity of innovative cold-formed steel columns. J.Constr. Steel Res. 2003; 59(4): 489-508.
[8] AS 4600. Cold Formed Steel Structures. Australia/New Zealand Standards 2005.
[9] Young B. Tests and design of fixed-ended cold-formed steel plain angle columns. J. Struct. Eng. 2004; 130(12).
[10] AISI Manual. Cold-formed Steel Design Manual. American Iron and Steel Institute 2016.
[11] Ellobody E, Young B. Behavior of cold-formed steel plain angle columns. J. Struct. Eng. 2005; 131(3).
[12] Young B, Ellobody E. Design of cold-formed steel unequal angle compression members. Thin-Walled Struct. 2007; 45(3): 330-38.
[13] Chantel YL. Experimental study of steel single unequal-leg under eccentric compression. J. Constr. Steel Res. 2011; 67(6): 919-928.
[14]Young B, Chen J. Column tests of cold-formed steel non-symmetric lipped angle sections. J. Constr. Steel Res. 2008; 64(7-8): 808-15.
[15] Vishnuvardhan S, Samuel Knight GM. Behavior of cold-formed steel single and compound plain angles in compression. Adv. Steel Constr. 2008; 4(1): 46-58.
[16] Zhou F, James B, Lim P, Young B. Ultimate compressive strength of cold-formed steel angle struts loaded through a single bolt.Adv. Struct. Eng. 2012; 15(9): 1586-95.
[17] Maia WF, Vieira LCM, Schafe BW, Malite M. Numerical and experimental investigation of cold-formed steel double angle members under compression. International Specialty Conference on Cold-Formed Steel Structures, Missouri, USA, October 24th– 25th, 2012.
[18] MacDonald M, Kulatunga MP. Finite element analysis of cold-formed steel structural members with perforations subjected to compression loading. Mech. Mechanical Eng. 2013; 17(2): 127-139.
[19] Shifferaw Y, Schafer BW. Cold-formed steel lipped and plain angle columns with fixed ends. Thin-Walled Struct. 2014; 80: 142-52.
[20] Landesmann A, Camotim D, Dinis PB, Cruz R. Short-to intermediate slender pin-ended cold-formed steel equal-leg angle columns: Experimental investigation, numerical simulations and DSM design. Eng. Struct. 2017; 132(1) 471-93.
[21] Ananthi GBG, Vishnuvardhan S, Samuel Knight GM. Experimental and numerical investigation on thin-walled single and starred angle sections under compression. Arab. J. Sci. Eng. 2015; 40: 3417–27.
[22] Silvestre N, Dinis P.B, Camotim D. Development on the design of cold formed steel angles. J. Struct. Eng. 2013; 139(5).
[23] Ananthi GBG. A study on cold-formed steel compound angle section subjected to axial compression. KSCE J. Civ. Eng. 2018; 22(5): 1803–18.
[24] Georgieva I, Schueremans L, Pyl L, Vandewalle L. Numerical study of built-up Double-Z members in bending and compression. Thin walled Struct. 2012; 60: 85-97.
[25] ASTM E8/E8M-13a. Standard test methods for tension testing of metallic materials. West Conshohocken, PA, 2013.
[26] IS 801: Code of Practice for Use of Cold Formed Light Gauge Steel Structural Members in General Building Construction. 2005.
[27] BS 5950 Part 5. Structural use of steelwork in building - Code of practice for design of cold formed thin gauge sections. 2002.