BEHAVIOUR OF COLD-FORMED STEEL SINGLE AND COMPOUND PLAIN ANGLES IN COMPRESSION

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Received: 28 November 2006; Revised: 4 May 2007; Accepted: 9 July 2007

ABSTRACT: Compression tests were carried out on cold-formed steel single angles, double angles welded back-to-back and starred angles under three different end connections. Eight plain sections of different sizes of two different thicknesses of different strengths were tested as stub and as short columns. The effect of flat width to thickness ratio, the effect of size of the section, the effect of material yield strength and the effect of symmetricity of the cross-section on the load carrying capacity is studied. Load versus axial shortening behaviour, load versus lateral deflection behaviour and load versus strain behaviour is also studied. The initial stiffness and ductility co-efficient for different sections are computed and compared. The ultimate loads of double angles welded back-to-back and starred angles with ball end connection is found to be more than twice when compared to single angles irrespective of the slenderness ratio. It was found that initial stiffness of double angles welded back-to-back is 1.25 times more than that of single angles. There was reversal of strain from compression to tension for compound angles beyond 70% of ultimate load. The effect of symmetricity on the mode of buckling of the specimens under different slenderness ratios is also reported.

Keywords: Cold-formed; single angle; compound angle; double angle; starred angle; stub column; short column; buckling

1. INTRODUCTION

Cold-formed steel structural members can lead to more economic design than hot-rolled members as a result of their high strength to weight ratio, ease of fabrication and construction. These sections are essentially thin-walled members with moderate to very high flat width to thickness ratio of the web or flange plate components. These members can be produced in a wide variety of sectional profiles such as angles, channels, hat sections, zed sections and sigma sections. Angles are the most common structural shape found in almost any structure due to their simplicity and ease of fabrication and erection. Single angles are usually used as web members in steel joists and trusses, members of latticed transmission towers or communication structures and bracing members to provide lateral support to the main members. Compound angles either as double angles or as starred angles or as built up sections are frequently used as main members in trusses and in transmission line towers.

Murray and Sherief [1] reviewed the Simple Column Design Practice followed in Canada and United States and Load and Resistance Factor Design of AISC Design Practice for single angle compression members attached by one leg. The failure loads obtained in an experimental study were compared with the compressive resistances calculated in accordance with these design approaches. Finite element results were also compared with the experimental results. It was concluded that the AISC approach reflects the behaviour of single angles correctly.

Compression tests on pin-ended and fixed-ended cold-formed in-line galvanised angle sections over various slenderness ratios taking into account detailed measurements of material properties, residual stresses and geometrical imperfections was conducted by Popovic, Hancock and Rasmussen [2]. It was found that the capacities of stub column tests are 15 and 40% higher than those calculated according to Australian and American specifications for cold-formed and hot-rolled steel structures. Popovic, Hancock and Rasmussen [3] conducted a series of compression tests on equal angles with slender cross-sections. The angles were tested between pinned ends and loaded axially with eccentric load, which caused bending parallel to the leg. The test data were compared with the design rules of the Australian and American specifications for cold-formed and hot-rolled steel structures, and as well as ASCE standards. The rules of the specifications for cold-formed steel structures are shown to be very conservative.

Young [4] conducted series of tests on cold-formed steel plain angle columns compressed between fixed ends. Tests were performed over a range of lengths such that column curves could be obtained. Geometric imperfections and material properties of the specimens were measured. The test strengths were compared with the design strengths calculated using the American specification and Australian/New Zealand standard for cold-formed steel structures. It was shown that the design strengths predicted by the specification and standard are generally very conservative. Modified design equations for cold-formed steel plain angle columns were proposed.

An efficient and accurate finite element model incorporating geometric imperfections and material non-linearities was developed by Ellobody and Young [5] to understand the behaviour of cold-formed steel plain angle columns. The finite element analysis was performed on plain angles compressed between fixed ends over different column lengths, and column curves were obtained. The column strengths predicted from the finite element model were compared with the design strengths calculated using the American specification and Australian/New Zealand standard for cold-formed steel structures. The results obtained from the finite element model were also compared with the design strengths obtained from the design rules proposed by other researchers.

2. EXPERIMENTAL INVESTIGATION

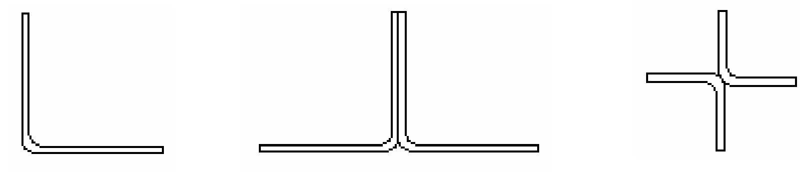
2.1 Test Specimens and End Connections

The specimens used in the present investigation were fabricated from different steel sheets of two different thicknesses 2.00 and 3.15 mm having different material properties. Tensile coupons were prepared and tested according to ASTM A 370 [6] to determine the yield stress, ultimate stress and percentage elongation. Table 1 presents the average material properties obtained from the tension tests.

Table 1. Tension Coupon Test Results

Type of Steel	Thickness (mm)	Modulus of Elasticity, MPa	Yield Stress (f_y), MPa	Ultimate Stress (f_u), MPa	f_u/f_y	% Elongation
1	2.00	182000	345	440	1.28	16
2	2.00	211000	415	495	1.20	10
3	3.15	201000	250	350	1.40	11
4	2.00	179000	205	300	1.46	13
5	2.00	210000	310	410	1.32	10
6	3.15	208000	250	365	1.46	26

Figures 1 (a) to (c) show the different cross-section geometries tested. Single angles of required sizes were obtained directly by brake-pressing, whereas the double angles welded back-to-back and starred angles were prepared by suitably welding the two single angles of same size confirming to AISI Manual - 1996 [7].



(a) Single Angle (b) Double Angle (c) Starred Angle

Figure 1. Cross-section Geometries of Angle Tested

Fifty seven experiments were conducted on single angles, double angles welded back-to-back and starred angles with three different end connections. The specimens were tested either as stub columns or as short columns. Thirty six specimens were tested as stub columns of slenderness ratio 10, 15 and 20. Stub columns are specimens whose height is not less than three times the largest dimension of the section and not more than twenty times the least radius of gyration as prescribed in the IS : 801 - 1975 [8]. Twenty one tests were carried on short columns of slenderness ratio 25 and 30. Table 2 shows the flat width to thickness ratio of the sections, limiting flat width to thickness ratio as per IS : 801 - 1975, type of steel from which the section is obtained, slenderness ratios of the specimens tested and the type of end connection. The sections chosen were those which are listed in IS : 811 - 1987 [9].

Table 2. Details of Test Specimens

Sl. No.	Section Size (mm)	(w/t)	$(w/t)_{lim}$	Type of Steel	Slenderness Ratios	End Connection
1	35 × 35 × 2.00	15.00	9.02	1	15, 20, 25	Ball
2	45 × 45 × 2.00	20.00	9.02	1	15, 20, 25	Ball
3	40 × 40 × 2.00	17.50	8.23	2	15, 30	Ball
4	40 × 40 × 3.15	10.20	10.60	3	15, 30	Ball
5	50 × 50 × 2.00	22.50	11.71	4	10, 15, 20	Welded
6	60 × 60 × 2.00	27.50	9.52	5	20, 30	Bolted
7	60 × 60 × 3.15	16.55	10.60	6	20, 30	Bolted
8	70 × 70 × 3.15	19.72	10.60	6	20, 30	Bolted

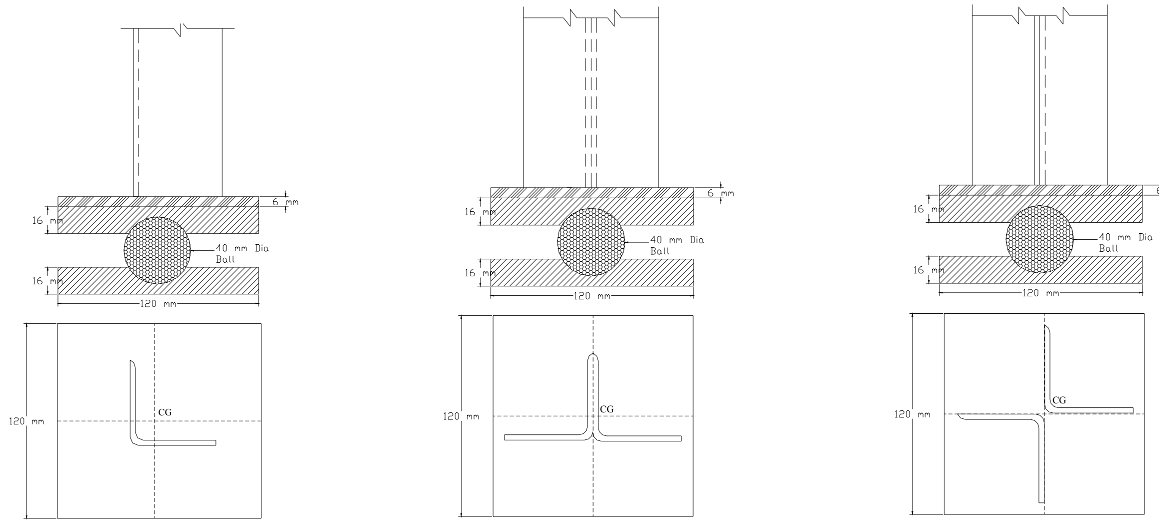
Three different end connections were used in the present investigation to study their effect on the load carrying capacity. Figures 2 to 4 show the details of three different end connections for single angles, angles welded back-to-back and starred angles.

2.1.1 Ball End Connection

This end connection is fabricated by welding the sections to an end bearing plate of 6 mm thick and in turn connected to a pair of flat bearing plates of sizes varying from 60 × 60 mm to 120 × 120 mm. The assembly consisted of two end plates of 16 mm thick with a spherical groove in between to accommodate a ball of 40 mm diameter at both ends.

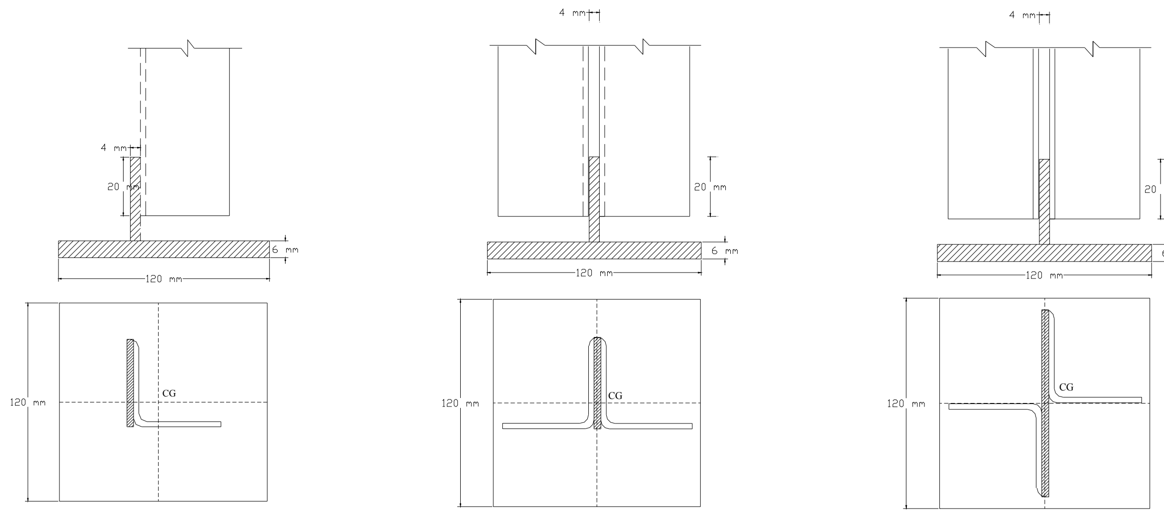
2.1.2 Welded End Connection

This end connection is fabricated by welding the section to the end plate of size varying from 60 × 60 × 6 mm to 120 × 120 × 6 mm through a gusset plate in such a way that the centre of the plate coincided with the centroid of the specimen. Then the section is tested with the load line coinciding with the centroid of the end plates.



(a) Single Angles (b) Double Angles (c) Starred Angles

Figure 2. Details of Ball End Connection

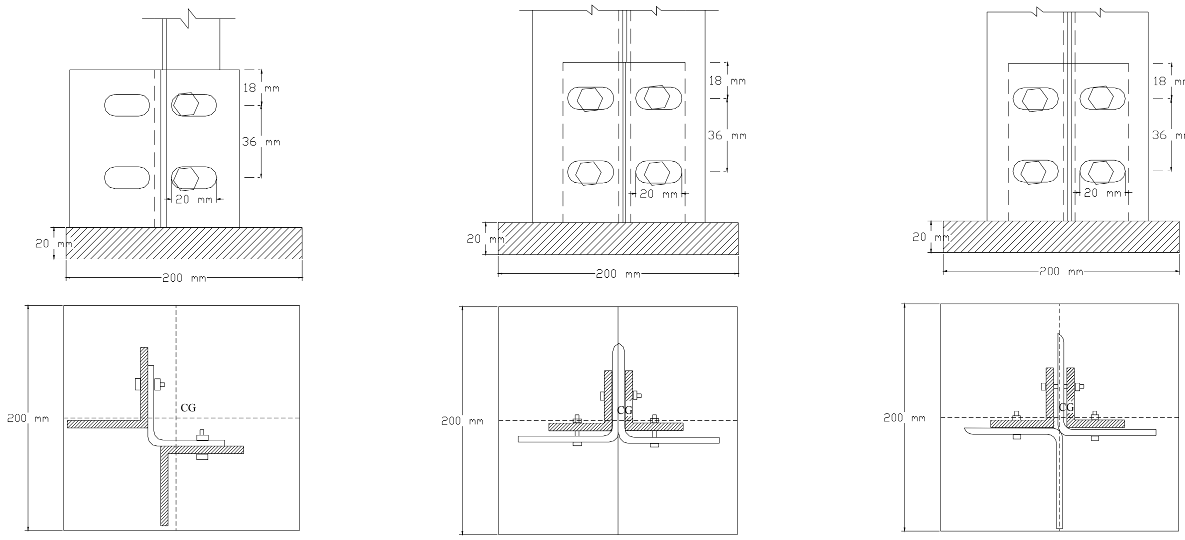


(a) Single Angles (b) Double Angles (c) Starred Angles

Figure 3. Details of Welded End Connection

2.1.3 Bolted End Connection

The specimens were bolted to two hot-rolled ISAs of size varying from 50 × 50 × 6 mm and 75 × 75 × 6 mm connected to 20 mm thick base plate of size 200 × 200 mm. The angles which are welded to the base plates were provided with a slot arrangement to accommodate specimens of different sizes and maintains the centre of gravity of specimens in line with the base plate. Bolt holes of 12 mm nominal diameter were made in the specimens to connect to the gusset angles confirming to AISI Manual - 1996.



(a) Single Angles (b) Double Angles (c) Starred Angles

Figure 4. Details of Bolted End Connection

2.2 Compression Test Set Up

Compression tests were carried out on a column testing machine. The test specimens were connected to the end fixtures and were tested to failure. For each test the axial load was increased at a relatively faster rate in the elastic range and at a slower rate in the plastic range till the specimen collapsed. Mitutoyo and Batty dial gauges of least count 0.01 mm were used to measure the axial shortening and lateral deflections of the member. Dial gauges were placed at the mid height of the section and also at one fourth of the height of section with their tips touching the web and flange of the specimens to measure the lateral deflections. One dial gauge was placed with its tip touching the movable head of the column testing machine to measure the axial shortening of the test specimen. Electrical resistance strain gauges of type BKSA 20 with the gauge factor of 2.1 ± 0.6 were used to measure the strains at mid-height. Strain gauges were fixed on flanges at mid height of the specimen. Care was taken that these strain gauges were fixed farther away from the neutral axis and to check whether the cross-sections experience symmetric or unsymmetric buckling. Strain indicator with 10 channels was used to record the strain measurements. The strain gauge and dial gauge readings were taken at higher intervals in the elastic range and at closer intervals in the inelastic range until the specimen failed. Figure 5 shows the test set up for bolted end connection.

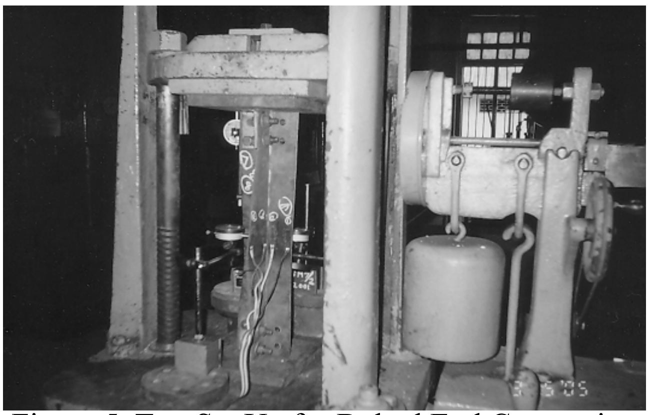


Figure 5. Test Set Up for Bolted End Connection

3. RESULTS AND DISCUSSION

The behaviour of cold-formed steel single angles, double angles welded back-to-back and starred angles under axial compression was studied. The load versus axial shortening behaviour, the load versus lateral deflection behaviour and the load versus strain behaviour under the elastic as well as in the plastic ranges of loading were studied. The effect of slenderness ratio, cross-section geometry of the angle, type of steel and the flat width to thickness ratio of the angle on the load carrying capacity were studied. The effect of symmetricity and the mode of buckling of the angles were also studied.

3.1 Load Carrying Capacity

The experimental ultimate loads obtained for single angles, double angles welded back-to-back and starred angles and also their ratios are presented in Table 3.

Table 3. Experimental Ultimate Loads of Single and Compound Angles and Their Ratios

Section Size (mm)	λ	Ultimate Load (P_{ex}), kN			$\frac{P_{double}}{P_{single}}$	$\frac{P_{starred}}{P_{single}}$
		Single Angles	Angles welded back-to-back	Starred Angles		
Ball End Connection						
35 × 35 × 2.00	15	32.03	76.09	76.32	2.3756	2.3828
	20	31.59	73.87	74.87	2.3384	2.3701
	25	29.37	69.42	72.09	2.3636	2.4545
45 × 45 × 2.00	15	40.05	80.28	83.39	2.0045	2.0821
	20	32.69	75.46	77.87	2.3084	2.3821
	25	27.02	70.49	54.05	2.6088	2.0004
40 × 40 × 2.00	15	48.47	69.14	66.74	1.4264	1.3769
	30	46.90	65.10	59.86	1.3881	1.2763
40 × 40 × 3.15	15	62.12	127.30	117.50	2.0493	1.8915
	30	59.20	123.50	115.30	2.0861	1.9476
Welded End Connection						
50 × 50 × 2.00	10	26.00	58.50	57.91	2.2500	2.2273
	15	25.75	55.00	56.05	2.1359	2.1767
	20	24.50	49.20	55.50	2.0082	2.2653
Bolted End Connection						
60 × 60 × 2.00	20	31.15	64.52	66.75	2.0713	2.1429
	30	28.49	59.30	61.65	2.0814	2.1639
60 × 60 × 3.15	20	68.97	127.71	123.70	1.8517	1.7935
	30	52.51	123.23	117.53	2.3468	2.2382
70 × 70 × 3.15	20	73.42	128.38	121.65	1.7486	1.6569
	30	58.29	101.65	117.72	1.7439	2.0196

It is observed that in the case of angles tested with ball end connection the increase in ultimate load of double and starred angles is twice when compared with single angles irrespective of the slenderness ratio. Similarly, the ultimate loads of double and starred angles are 1.90 times more as compared to single angles with bolted end connection of slenderness ratio less than 20. For short columns the increase is found to be 2.05 and 2.15 for double and starred angles, respectively. It is observed that in the case of angles tested with ball end connection there is only a marginal decrease in ultimate loads due to increase in the slenderness ratio from 15 to 30, irrespective of the type of angle whether single or compound. Similarly for angles tested with welded end connection the increase in ultimate loads is insignificant due to increase in slenderness ratio from 10 to 20. In the case of angles tested with welded end connection the increase in slenderness ratio from 15 to 30 decreases the ultimate load carrying capacity by 15% in the case of single angles and 10% in the case of double and starred angles. For the angles with bolted end connection the decrease in ultimate load is 16% for single angles, 8% for double and starred angles when the slenderness ratio is increased from 20 to 30.

3.2 Load Versus Axial Shortening Behaviour

The load versus axial shortening behaviour is initially linear in most cases up to 50% of ultimate load irrespective of the cross-section of the specimens. Figures 6 and 7 show the typical load versus axial shortening behaviour of stub and short angles with ball end connection. It is observed that the behaviour is steeper for stub columns as compared to short columns. Similarly for angles with welded end connection there is a sudden drop in the load after the ultimate load and the non-linearity in the behaviour starts at 70% of ultimate load for all angles.

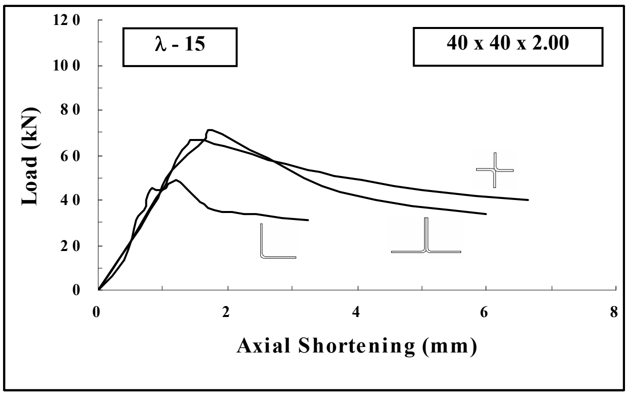


Figure 6. Axial Shortening Behaviour of StubAngles with Ball End Connection

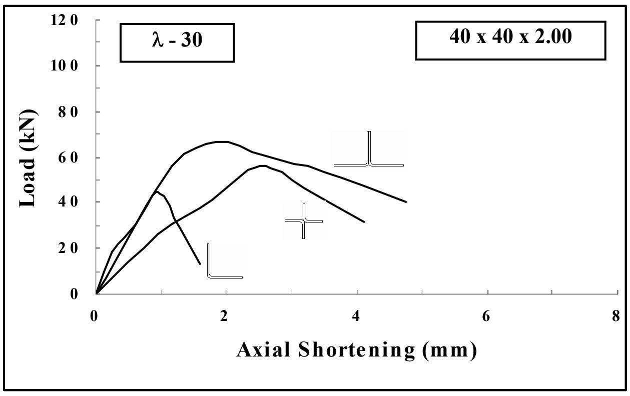


Figure 7. Axial Shortening Behaviour of Short Angles with Ball End Connection

Figures 8 and 9 show the typical load versus axial shortening behaviour of stub and short angles with bolted end connection. It is observed that the behaviour is similar irrespective of the cross-section and slenderness ratio. The long horizontal plateau after the ultimate load is noticed, indicates high degree of ductility. For specimens with higher flat width to thickness ratios the non-linearity starts at 40% of the ultimate load whereas for flat width to thickness ratio less than the limiting, the non-linearity starts at 50% of the ultimate load. Flat width to thickness ratio certainly plays a significant role in the load carrying capacity.

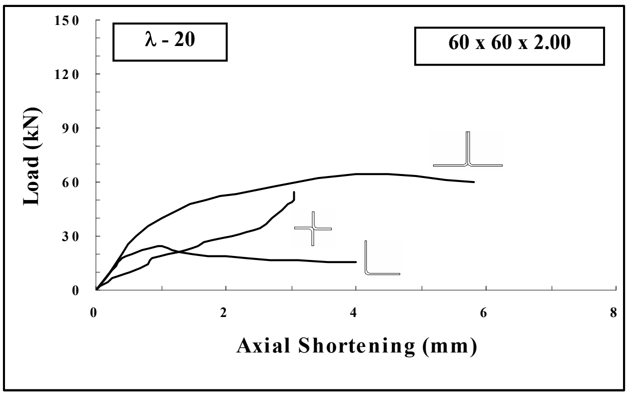


Figure 8. Axial Shortening Behaviour of Stub Angles with Bolted End Connection

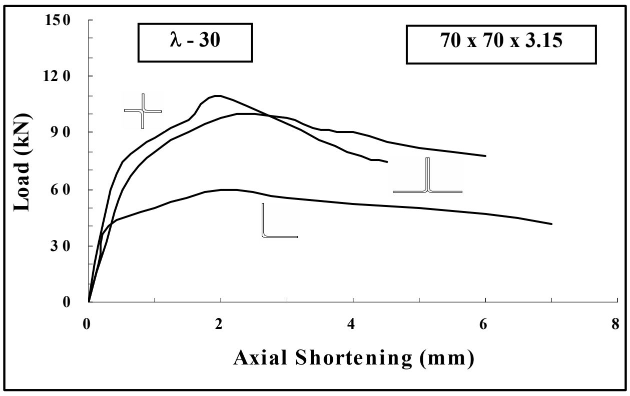


Figure 9. Axial Shortening Behaviour of Short Angles with Bolted End Connection

The initial stiffness is calculated for all the sections tested from the load versus axial shortening behaviour. In the case of angles tested with ball end connection it is observed that the initial stiffness of stub angles was always less than short angles by 25%, irrespective of the configuration of the section. The stiffness of single angles is higher compared to double and starred angles when tested as stub or as short columns. For angles tested with welded end connection, the stiffness of starred angles is over two times when compared to single angles and over 1.3 times as compared to double angles when tested as stub columns. Similarly, in the case of angles tested with bolted end connection, for single and starred angles the initial stiffness of stub columns is higher than that of short columns. For double angles the initial stiffness of short columns is higher than stub columns by 15%.

To quantify the ductility of stub and short columns under different end connections, ductility co-efficient μ as defined by Han, Zhao and Tao [10] is adopted. It is calculated as the ratio of the axial displacement (Δ_u) corresponding to 85% of ultimate load in the post ultimate region to the axial displacement at yield load (Δ_y). For angles tested with ball end connection, the co-efficient is more for starred angles as compared to single and double angles tested as stub columns by 15 and 45%, respectively. Similarly for angles tested as short column the ductility co-efficient is more for double angles as compared to single and starred angles by 20 and 45%, respectively. For angles with welded ends, the ductility co-efficient of double angles is more as compared to single and starred angles. It is observed that the ductility co-efficient of angles tested with bolted end connection was generally higher than that for angles tested with ball end as well as with welded end connection. The ductility co-efficient of short columns is more as compared to stub columns irrespective of the angle whether single, double or starred.

3.3 Load Versus Lateral Deflection Behaviour

The lateral deflections were observed to be meager up to 75% of ultimate load and beyond which the magnitude increases enormously indicating buckling of the flanges for specimens with ball ends. Whereas for angles tested with bolted ends the non-linearity in behaviour starts from the onset of loading. The lateral deflection of connected leg is higher than that of unconnected leg. In most of the cases the lateral deflection behaviour is found to be symmetrical with respect to both the legs indicating that the load transfer is through the axis of the angle. It is observed in general, that lateral deflection is predominant only after 60% of the ultimate load. There is a gradual increase in the elastic limit and at the attainment of the plastic stage there is a sharp increase in the lateral deflection. The load versus lateral deflection behaviour of double angles is distinctively different with those of single angles which have steep initial slope.

3.4 Load Versus Strain Behaviour

Figures 10 to 12 show the typical load versus strain behaviour of single angles, double angles welded back-to-back and starred angles with different end connections. In the case of single angles tested with welded ends, it is observed that the portion near the root of the unconnected leg buckled first, and remained compressive throughout. The toe end of the connected leg remained compressive in the initial stage and changed over to tension near the ultimate load, and remained tensile throughout, for all the slenderness ratios.

For double angles, at the toe end of the unconnected legs the compressive strains changed over to tensile strains at 75% of ultimate load. The toe end of the connected legs remained compressive till the failure of the specimen. In the case of starred angles almost the entire cross-section changed over to tension at 70% of ultimate load indicating a twist in the cross-section. The angle failed even before reaching the yield.

In the case of single angles tested with bolted ends, it is observed that the non-linearity in the strain behaviour started much earlier than yield strain. In single angles tested as stub column both compressive and tensile strains prevailed in the legs until failure. For single angles tested as short column, the strains measured at the corner of one of the legs remained tensile and in other locations remained compressive.

For double angles tested as stub column the strains in most of the locations remained within the yield strain irrespective of the configuration of the section. The non-linearity in the behaviour started at 75% of ultimate load. The strains increased more steeply with the increase in load in the case of double angles tested as short columns compared to stub columns. In starred angles when tested as stub column the strains were tensile in all the legs except in one leg, where tensile strain changed over to compressive at 40% of the ultimate load. Similarly in starred angles tested as short columns the strains were initially compressive and later changed over to tensile.

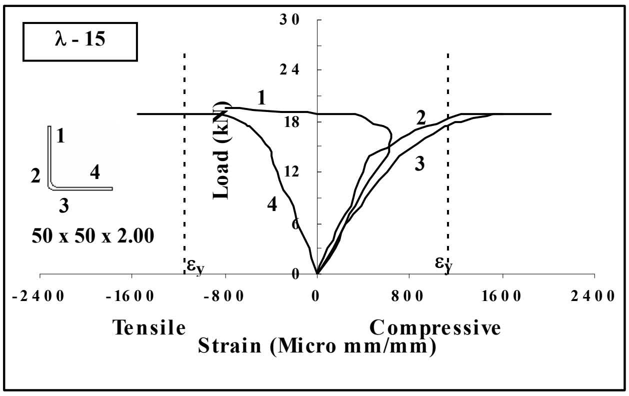


Figure 10. Load Versus Strain Behaviour of Single Stub Angles with Welded End Connection

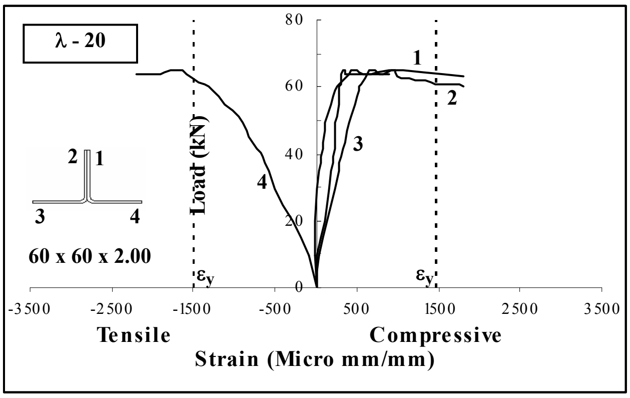


Figure 11. Load Versus Strain Behaviour of Double Stub Angles with Bolted End Connection

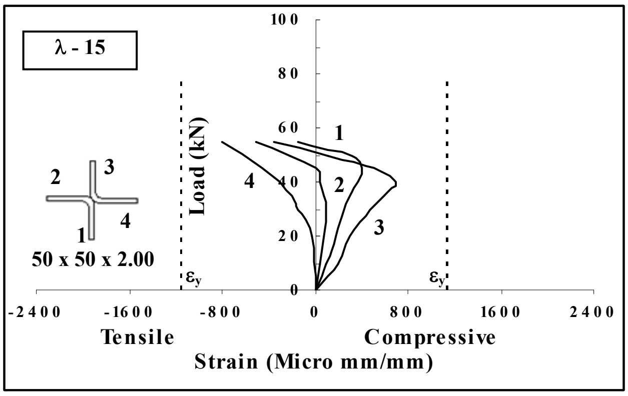


Figure 12. Load Versus Strain Behaviour of Starred Stub Angles with Welded End Connection

3.5 Failure Modes of Angles

The mode of failure of all the specimens tested with three different end connections were noticed during testing. The angles failed either by local plate buckling, flexural buckling about the weak axis, torsional-flexural buckling or torsional buckling. The failures were distinctly different for singly symmetric sections and doubly symmetric sections. The mode of failure and the location of failure depends on the slenderness ratio, cross-section and symmetricity of the section. The different failure modes of the angles tested under different end connections are discussed. Local plate buckling was observed in the case of single angles tested with ball end connection. The local buckling occurred at mid height of flange or between mid height and one third of flange of the angle irrespective of the section whether it is stub or short column. Local plate buckling was also noticed in the flanges of stub and short columns of single angles with welded end connection.

Figure 13 shows the failure of single angle by local buckling. In the case of starred angles, local plate buckling was noticed when tested as short columns at mid height of the section with welded end connection. Similarly stub columns tested as double angles welded back-to-back with bolted ends predominantly failed by local buckling caused either at the end of the section or at one-fourth height of the section. In the case of single angles tested as stub columns with bolted end connection, most of the sections failed by local plate buckling initiated either at the mid height or at one-fourth height or at the ends.

Figure 13. Failure of Single Angle by Local Plate Buckling under Bolted End Connection

Failure of double angles welded back-to-back with ball end was also failed by flexural buckling irrespective of the slenderness ratio of the section. The failure of these sections occurred always in the flanges of the sections either at mid height or at one-third height. Single angles tested as short column with bolted end connection failed by flexural buckling at one-fourth height of the section or by local plate buckling initiated at the end of the section. In the case of short columns of double angles and stub columns of starred angles the failure is by flexural buckling. Torsional-flexural buckling was noticed in most of the double angles. It is observed that failure is caused either by flexural or torsional buckling in the case of starred angles tested with ball end connection. The failure occurred between the mid and one-third height of the section in the flanges. Starred stub angles with welded ends failed by flexural buckling. In the case of starred angles tested as short columns with bolted end connection the failure is by torsional buckling. Figure 14 shows the torsional buckling failure of starred angle.

Figure 14. Torsional Buckling of Starred Angle with Bolted End Connection

4. CONCLUSIONS

Based on the experimental investigation carried out on the compression behaviour of single angles, double angles welded back-to-back and starred angles, the following conclusions are drawn.

- In the case of angles tested with welded end connection the increase in slenderness ratio from 15 to 30 decreases the ultimate load carrying capacity by 15% in the case of single angles and 10% in the case of double and starred angles.

- For angles with welded end connection the ultimate loads of double and starred angles are 2.10 and 2.20 times more than that of single angles when tested as stub columns.

- For specimens with higher flat width to thickness ratio non-linearity in axial shortening behaviour starts at 40% of the ultimate load whereas for smaller flat width to thickness ratio non-linearity starts at 50%. For stub columns the behaviour is steeper as compared to short columns.

- The initial stiffness of starred angles is twice as compared to single angles and one and a half times more as compared to double angles when tested as stub columns with welded end connection.

- The ductility co-efficient of angles with welded ends is 50% more as compared to angles with ball ends. Similarly the ductility co-efficient of angles with bolted ends is twice as compared to angles with ball ends.

- For double angles with welded ends the compressive strains at the toe end of the unconnected legs changed over to tensile strains at 75% of ultimate load.

- Double angles welded back-to-back with bolted ends failed by local buckling which occurred at the end of the section or at one-fourth the height and starred angles with welded ends also failed by local plate buckling at mid height of the section.

- Single angles tested as short column with bolted ends failed by flexural buckling at one-fourth the height of the section and starred angles tested as short column with bolted ends failed by torsional buckling.

REFERENCES

[1] Murray, C. Temple and Sherief, S.S. Sakla, "Consideration for the Design of Single Angle Compression Members Attached by One Leg", Proceedings of Structural Stability and Design, Rotterdam, 1995, pp. 107-112.

[2] Popovic, D., Hancock, G.J. and Rasmussen, K.J.R., "Axial Compression Tests of Cold-formed Angles", Journal of Structural Engineering, ASCE, 1999, Vol. 125, No. 5, pp. 515-523.

[3] Popovic, D., Hancock, G.J. and Rasmussen, K.J.R. (2001), "Compression Tests of Cold-formed Angles Loaded Parallel with a Leg", Journal of Structural Engineering, ASCE, Vol. 127, No. 6, pp. 600-607.

[4] Young, B., "Tests and Design of Fixed-Ended Cold-Formed Steel Plain Angle Columns", Journal of Structural Engineering, ASCE, 2004, Vol. 130, No. 12, pp. 1931-1940.

[5] Ellobody, Ehab and Young, Ben, "Behaviour of Cold-formed Steel Plain Angle Columns", Journal of Structural Engineering, ASCE, 2005, Vol. 131, No. 3, pp. 457-466.

[6] ASTM : 370 - 92, "Standard Test Methods and Definitions for Mechanical Testing of Steel Products", 1996.

[7] AISI Manual, "Cold-formed Steel Design Manual", American Iron and Steel Institute, 1996.

[8] IS : 801, "Code of Practice for Use of Cold-formed Light Gauge Steel Structural Members in General Building Construction", Bureau of Indian Standards, 1975.

[9] IS : 811, "Specification for Cold-formed Light Gauge Structural Steel Sections", Bureau of Indian Standards, 1987.

[10] Han, L.H., Zhao, X.L. and Tao, Z., "Tests and Mechanics Model for Concrete-filled SHS Stub Columns, Columns and Beam-columns", Steel and Composite Structures, 2001, Vol. 1, No.1, pp. 51-74.