Study on the Behavior of Cold-formed Steel Angle Tension Members

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STUDY ON THE BEHAVIOUR OF COLD-FORMED STEEL ANGLE TENSION MEMBERS

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ABSTRACT

Cold-formed steel tension members with bolted end connections are frequently used in a variety of structures such as trusses, transmission towers etc. Among all the shapes, angles are widely used. When angle sections are connected with gusset plates and eccentrically loaded, their ultimate load-carrying capacity is influenced by the effect of shear lag. This paper presents the details of an experimental and numerical investigation with a primary objective of studying the effect of shear lag on cold-formed steel single and double angles subjected to tension. Seventy-two single plain and lipped angles made from thicknesses 2,3 and 4 mm connected to gusset plates at their ends by ordinary black bolts were tested. Forty-eight double angles of 3 and 4 mm thicknesses connected to the opposite side of gusset plate and to the same side of the gusset plate at their ends by black bolts were also tested. All the one hundred and twenty specimens were tested in an Universal Testing machine subjected to eccentric tensile load. From the test results, load vs deflection behaviour and the failure modes were studied. The actual load carried by the specimen was compared with the theoretical load carrying capacity predicted by International codal provisions and with the load carrying capacity predicted by numerical investigation by ANSYS. An empirical equation is proposed to determine the load-carrying capacity of the cold-formed steel angles and the predicted values agree with the experimental results.

INTRODUCTION:

Cold- formed steel structural elements are widely used as structural elements in roofs, decks, wall panels, trailer bodies, agricultural equipments, aircrafts, etc. Angles are the most basic and widely used sections among the various forms of all rolled steel sections available. Practically angles are connected with gusset plates through one leg and due to this there will be non-uniform stress distribution due to eccentrically applied load. **Chesson and**

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Munse carried out the study of shear lag effects on single and double angles made of hot rolled sections. Their study included different cross-sectional configurations, connections, materials and fabrication methods. The theoretical concept of shear lag and its effect on the angle members were based on test results of 218 specimens (among which there were 137 angle specimens) of various configurations. **Chi – Ling Pan** conducted tests on cold formed steel channel sections with different dimensions to investigate the effect of shear lag. The comparisons were made between the test results and predictions computed based on several specifications. To study the stress distribution at the various locations of the cross-section of specimen, the finite element software ANSYS was used. **Epstein and Chamarajnagar** formulated a 20 node quadratic brick element model. The material nonlinear effects were modeled using the Von-Mises yield criterion and the material stress-strain curve was assumed to be elastic perfectly plastic. **La–Boube and Yu** conducted an experimental and analytical study at the University of Missouri–Rolla, to expand the knowledge and understanding of the behaviour of cold-formed steel bolted connections. The first part concentrated on the tensile capacity, bearing capacity and the interaction of tension and bearing capacities of flat sheet cold-formed steel bolted connections. In the second part, the tensile capacity and bearing capacity of bolted connections of flat sheet, angle and channel cold-formed steel members were addressed. **Mohan Gupta and L.M.Gupta** analyzed angles with bolted connections using Finite Element method giving due considerations to associated problems such as the shape of the material, stress-strain curve, the contact between the gusset plate and the angle, the appropriate failure criteria, the effect of punching of holes etc. He also analysed angles under tension in the limit state format giving due considerations to block shear failure and yielding of gross section. The factor of safety obtained as a result indicated adequate representation of design strengths. **Wu and Kulak** conducted an experimental investigation of single and double angle tension members to examine the effect of shear lag on the net section rupture capacity of the cross section. They tested 24 specimens (11 single angle members and 13 double angle members) to compare the ultimate loads with the earlier test results obtained by others. They also conducted finite element investigation to determine the stress distribution of the critical cross section at ultimate load. **Valdier Francisco de paula et al,** presented experimental results of 66 specimens carried out on cold-formed steel angles fastened with bolts under tension. He conducted multiple linear regression analysis and suggested the expression for net section efficiency (U) which depended on the geometrical factors such as connection eccentricity ($\overline{x}$), connection length (L), width of connected leg of the angle (b_c), net width of the angle with connected leg (b_{cn}), width of unconnected leg (b_d), nominal bolt diameter (d) and angle thickness (t).

All the above investigations were made for the hot rolled double angle sections. There were only limited investigations for cold-formed steel members. The present investigation aims to study the behaviour of cold-formed steel angle members.

CODAL PROVISIONS

The existing Indian Standard code of practice for cold-formed steel IS 801-1975 does not elaborately deal with the design of tension members. The following codal provisions are used to predict member capacities of the cold-formed steel angle members.

American Iron and Steel Institute : Appendix A of North American specification, 2007 Edition

The nominal tensile strengthe strength P_n of the member,

$P_n = A_e F_u$

where A_e = UA_n and $U = 1.0 – 1.20\ \bar{x} / L < 0.9$ but shall not be less than 0.4

A_e = effective net area of the section

A_n = net area of the connected part.

$\bar{x}$ = distance from shear plane to centroid of the cross section.

L = length of the end connection i.e. distance between the outermost bolts in the joint along the length direction.

Australian/NewZealand Standards: AS/NZS 4600-2005

The nominal section capacity of a member in tension shall be taken as the lesser of

$N_t = A_g f_y$ and

$Nt = 0.85\ K_t A_n f_u$

where A_g = gross cross sectional area of the member

f_y = yield stress of the material

K_t = correction factor for distribution of forces.

for eccentrically connected single angles and double angles connected to opposite side of the gusset plate, the value of $K_t = 0.85$

for double angles connected to the same side of the gusset plate the value of $K_t = 1.0$

A_n = net area of the cross-section, obtained by deducting from the gross area of the cross-section, the sectional area of all penetrations and holes, including fastener holes.

f_u = tensile strength used in the design.

British Standards: BS:5950 (Part 5)-1998

The tensile capacity P_t, of a member

$P_t = A_e * p_y$

Single angles

For single angles connected through one leg only, the effective area A_e is computed as

$$A_e = a_1(3a_1+4a_2)/ (3a_1+a_2)$$

Double angles

For double angles connected to opposite side of gusset plate, the effective area is determined as

$$A_e = a_1(5a_1+6a_2)/ (5a_1+a_2)$$

For double angles connected to the same side of gusset plate the effective area can be determined as that of single angles.

A_e = effective area of the section

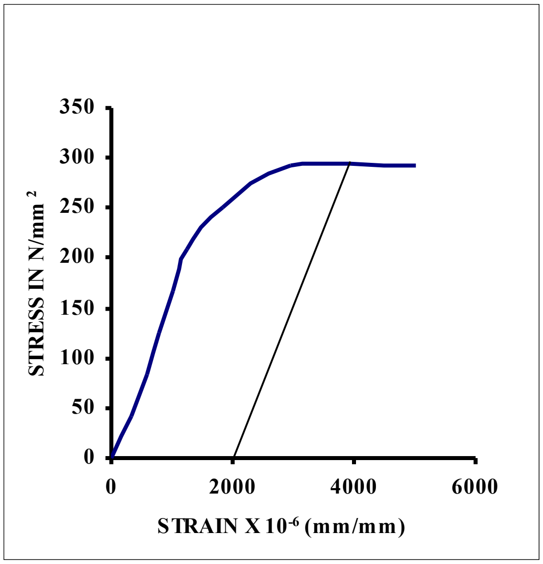
a_1 = the net sectional area of the connected leg

a_2 = the gross sectional area of the unconnected leg

p_y = the design strength.

EXPERIMENTAL INVESTIGATION

A total of one hundred and twenty experiments using seventy-two single angle specimens with and without lips of 2, 3 and 4mm thickness and twenty-four double angle specimens connected back to back side of the gusset plate of 3mm and 4mm thickness and twenty-four double angle specimens connected to the same side of the gusset plate of 3mm and 4mm thickness with bolted connections were conducted under eccentric tensile loads. The specimens were fabricated from 2mm, 3mm, 4mm thickness cold- formed steel sheets of grade St- 34-1079 by bending and press breaking operations. Standard tension tests were conducted on coupons, stress vs strain curve was plotted as shown in fig 1. The values of yield stress, ultimate stress, modulus of elasticity and elongation obtained for these thicknesses of cold formed steel sheets are presented in the Table 1.



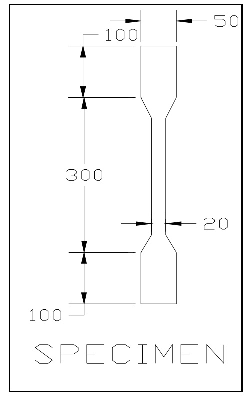


Fig 1 Standard tension test

Table 1. MECHANICAL PROPERTIES OF STEEL SHEET

Thickness of steel sheet in mm	Yield Stress in MPa (f_y)	Ultimate Stress in MPa (f_u)	Modulus of Elasticity in MPa	f_u/f_y	Percentage elongation in 200mm gauge length
2	210	268	2.00×10^5	1.27	10
3	228	292	2.00×10^5	1.28	11
4	235	313	2.03×10^5	1.33	14

The specimens were tested as two different section configurations namely single angles and double angles. The single angle specimens were connected with their larger leg to end gusset plates of mild steel of 6mm thickness. Ordinary black bolts of 12mm diameter are used as connectors for specimens made from 2mm and 4mm thickness sheets. In case of specimens fabricated from 3mm thickness sheet 10mm diameter bolts were used. The double angle specimens were connected with their larger leg with two mild steel gusset plates of 8mm and 12mm thickness using ordinary black bolts of 10mm and 12mm diameter. The gusset plates were not reused for single angle specimens and were reused for double angle specimens. The required number of bolts are calculated for all specimens and were provided according to the design procedures. All the specimens were fabricated for a length of 500mm. The width of the gusset plate was kept 10mm more than the width of the connected leg. The length of gusset plate was provided according to the requirement of pitch and edge distance as per Indian code of practice. All the members were connected with gusset plate to the larger side by means of bolts.

Fig 2,and 3 present the details of the fabricated single and double angle specimens. The specimens were tested in Universal Testing machine of 400kN and 1000KN capacity. The specimens were fixed vertically by gripping the gusset plates. The load was applied eccentrically through the gusset plates. Demec gauge was used for measuring the elongation for a gauge length of 200mm. The experimental set up is shown in fig 4,5,6. Figure 7 shows the gusset plates used for the connection. The load is gradually applied with suitable increments from control panel and at each increment of loads corresponding elongation was taken. The yield, ultimate and breaking loads were also observed. The distance of separation between gusset plate and test specimens was also recorded. The procedure is repeated till the failure stage is reached in all specimens. The observed yield load and ultimate load of the specimens tested are recorded.

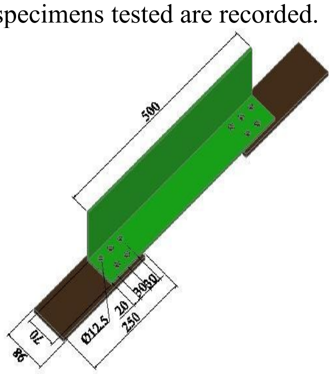


Fig 2 Details of single plain angle specimen provided with bolts in staggered pitch

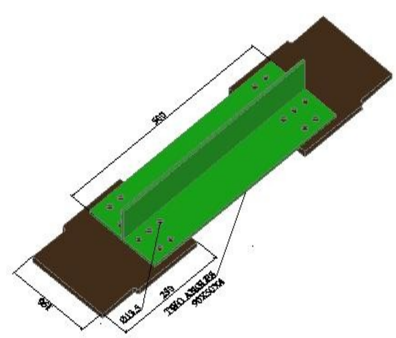


Fig 3 Details of double angle specimens connected to same side provided with bolts in staggered pitch.

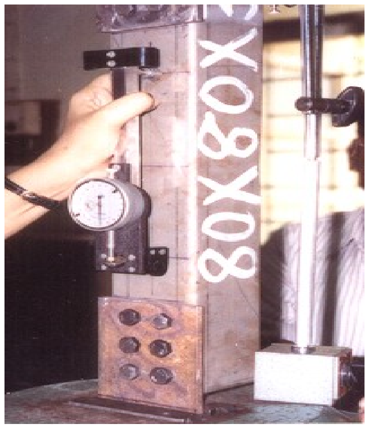


Figure 4 Experimental set up for single angle specimen

Figure 5 Experimental set up for double angles connected to opposite side of gusset plates

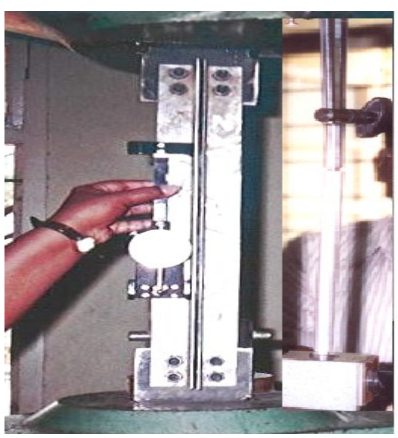
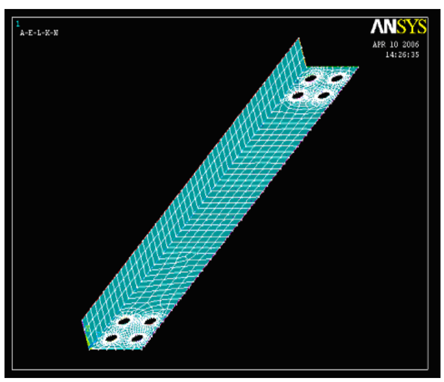


Figure 6 Experimental set up for double angles connected to same side of gusset plates

Figure 7 Gusset plates used for connection.

NUMERICAL INVESTIGATION:

The goal of the finite element analysis is to develop a model that could study the behaviour of bolted cold-formed steel single and double angle tension members. The behaviour observed during the tests was used for preparing a finite element model. All the one hundred and twenty specimens were modeled using the finite element program ANSYS (version 10). The problem was studied as a nonlinear load vs displacement analysis including plasticity and nonlinear effect of geometry. SHELL 63 element type was used to model the single and double angle specimens. It is a 4 noded 3-dimensional quadratic elastic shell element. It has both bending and membrane capabilities. This element has six degrees of freedom at each node: translations in the nodal x,y and z directions and rotations about the nodal x,y and z axes. A typical mesh of the model is shown in fig. 8,9. In the finite element models, the shear deformation of the bolts was ignored. The load was assumed to transfer from gusset plate to the angle fully by the bearing of the bolts.



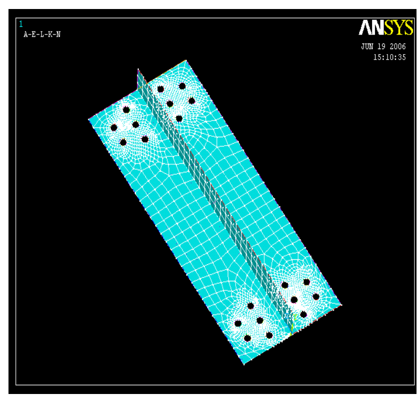


Figure 9 Element mesh for double unequal angle 100x50x3 connected to the same side of gusset plate

RESULTS AND DISCUSSION

The behaviour of cold-formed steel single and double angles when subjected to eccentric tension were studied. The ultimate-load carrying capacities of the specimens were compared with the load carrying capacities predicted using the American, Australian/New Zealand and British standards.

The experimental results were also compared with the numerical results obtained using ANSYS software and with the proposed equation for tensile load carrying capacity obtained using the nonlinear regression analysis.

1) EXPERIMENTAL INVESTIGATION

a) Ultimate Load-Carrying Capacity

The experimental ultimate loads for all the cold-formed steel single angles are presented in Table 2. It is observed that in the case of single equal lipped angles the average increase in ultimate load is 1.2 times greater than that of single equal plain angles. In the case of single unequal lipped angles the average increase in ultimate load is found to be 1.24 times greater than that of single unequal plain angles. The average increase in ultimate load for double equal angles connected to opposite side of the gusset plate is 1.25 times greater than that of double equal angles connected to the same side of the gusset plate. In the case of double unequal angles connected to oppostie side of the gusset plate the average increase in ultimate load is 1.27 times greater than that of double unequal angles connected to the same side of the gusset plate.

Table 2 Ultimate load carrying capacity of the single angles

S. No.	Size of the specimen (mm)	Ultimate load carrying capacity (P_{exp}) in kN		
		t = 2mm	t = 3mm	t = 4mm
1	40×40×t	25	47.5	64
2	50×50×t	33	55	80.5
3	60×60×t	47.5	69	83
4	70×70×t	55	82	92
5	80×80×t	60	96	123
6	40×25×t	19	33	42.5
7	50×25×t	28.5	41.5	59.5
8	60×30×t	36	43.5	62.5
9	60×40×t	39	50.5	68
10	80×30×t	41	68	80
11	90×50×t	51	88	101
12	100×50×t	66	98	125
13	40×40×15×t	34	59	75
14	50×50×15×t	43	63	98

15	60×60×15×t	57.5	71	103
16	70×70×15×t	60	109	135
17	80×80×15×t	72	124	137
18	40×25×15×t	26	57	64.3
19	60×25×15×t	40	60	70
20	60×30×15×t	43	62	75.5
21	60×40×15×t	48	63	83
22	80×30×15×t	56	74	90
23	90×50×15×t	60	86	120
24	100×50×15×t	65	106	125

b) Load vs Deflection

Figures 10 and 11 show the typical load versus deflection behaviour for single angles with and without lips and double angles. From the graphs, it is observed that the ultimate load carrying capacity increases as the cross-sectional area and number of bolts in the connection increases. It is also observed that when the rigidity of the connection increases the stiffness of the member also increases.

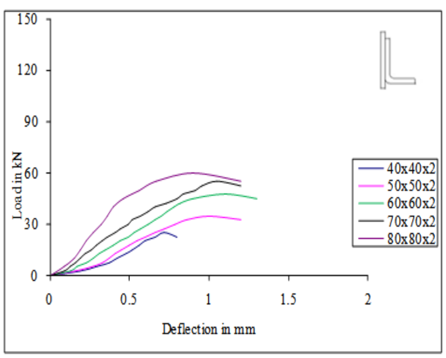


Fig 10 Load Vs Deflection for single equal plain angle thickness 2mm

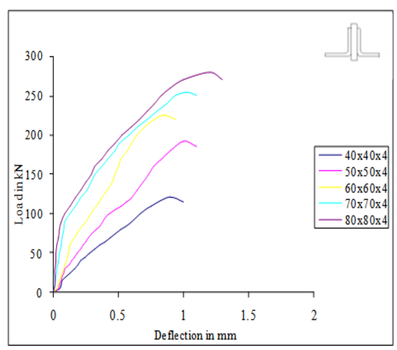


Fig 11 Load vs Deflection for double equal angle (opposite side), thickness 4mm

c) Modes of Failure

The mode of failure of all single and double angle specimens were noticed during testing. Generally tearing failure, block shear failure, net section fracture failure were observed as in fig 12,13, and 14 . The failure modes are different for single and double angle sections. The mode of failure depends upon the cross section and rigidity of connection.

During the loading process, the gusset plates of double angle members remained straight. However, in the case of single angles the gusset plate and the angles bent during loading. This is due to eccentrically applied load. This kind of bending is referred as global bending. As the load was being applied, the corners of the angle at the two ends gradually separated from the gusset plates for both single and double angle members. Thus, a gap was formed between the corner of the connected leg and the gusset plate. This is referred as local bending. The visible length of gap was usually from the edge of the angle to the innermost bolt. The width of the gap varied from one specimen to another, with a maximum observed value of 10mm. Generally larger gaps were associated with the cases of greater eccentricity of the cross-section, smaller angle thicknesses and shorter connection lengths.

There was no major slip of the connections during the tests. All the specimens failed at the critical cross-section (inner most bolt hole) as the ultimate load was reached. After necking, the critical cross-section was torn out from the edge of the connected leg to the hole then to the corner of the angle. The specimens carried some amount of load beyond the ultimate load and until failure. It was noted that all the bolts were still tight after completion of the tests. This indicates that the bolts were not highly stressed during the tests. The outstanding leg which is subjected to compression experiences local buckling nearer to the supports.

Fig 12 Tearing failure of lipped angle 60×25×15×3

Fig 13 Block shear failure of single plain angle 40x25x3

Fig 14 Net section fracture failure of plain angle 90×50×3

2) COMPARISON OF EXPERIMENTAL AND PREDICTED ULTIMATE LOADS

A comparative study between the experimentally observed ultimate loads of the specimen tested with the tensile load carrying capacity of equations of the following codes North American Specification-2007, AS/NZS:4600-2005, BS:5950 (Part 5)-1998 is made to review the the procedures recommended.

The comparison of predicted ultimate loads by the three various codes for single and double angles tested are shown in Figures 15 and 16. The tensile capacity equations of the international codes take it into account the effect of shear lag and incorporates the capacity reduction factor in addition to net effective area of the section. In case of single angles the values predicted by AISI and AS/NZS are nearly 11% lower than the ultimate loads irrespective of whether the angle is equal or unequal and provided with or without lip. BS code underestimates the values by 29% with respect to experimental ultimate loads. Provision of lip increases the load carrying capacity of the angles by 22%.

In case of double angles the ultimate loads predicted by the AISI and AS/NZS are nearly 20% lower than the experimental ultimate loads. BS code underestimates the values for double angles by 21%. It is also observed that the load carrying capacity for double angles connected to opposite side of the gusset plate is 26% more than that of double angles connected to the same side.

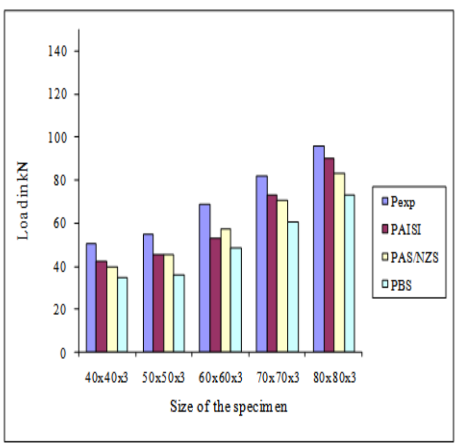


Fig 15 Comparison of ultimate loads with loads based on codal provisions for single equal plain angles

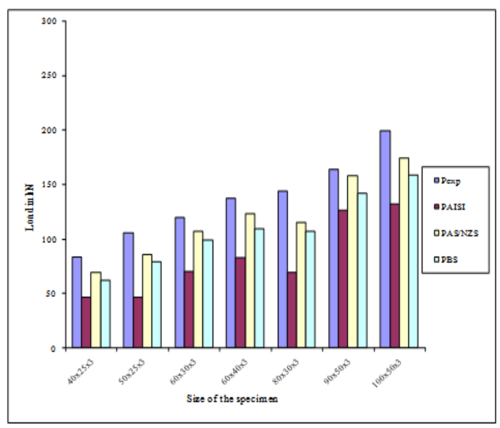


Fig 16 Comparison of ultimate loads with loads based on codal provisions for double unequal angles connected to opposite side of the gusset plate

3) NUMERICAL INVESTIGATION

To perform the non-linear analysis, the angle specimens are modeled based on the experimental set up incorporating geometric imperfections. The geometric imperfections included the thickness of the section, width of the connected leg, width of unconnected leg in case of single plain angles and it includes width of lip in case of lipped angles. As the nonlinear problem is path dependant, the solution process requires a step by step load incremental analysis. In the analysis, the solution usually converged very slowly after yielding, and the increment for each load step had to be made very small. Yielding is determined using von-Mises yield criteria. At the completion of each incremental solution, the program adjusts the stiffness matrix to reflect the nonlinear changes in structural stiffness before proceeding to the next load increment. ANSYS employs Newton-Raphson equilibrium iterations. The general post processor in ANSYS is used to review results at each load increments. Fig 17 and 18 shows the stress distribution.

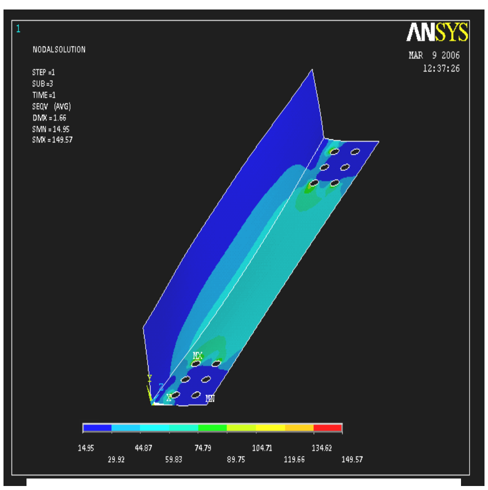


Fig 17 Stress distribution for single plain equal angle 80×80×3

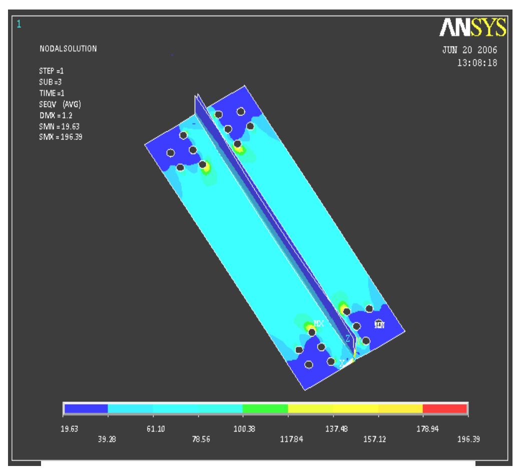


Fig 18 Stress distribution for double angle 100×50×4 connected to the same side of the gusset plate

4) PROPOSED EQUATION FOR PREDICTING STRENGTH DUE TO NET SECTION FRACTURE FAILURE.

The tensile strength of the angle sections can be evaluated in terms of the ratio of its average stress at ultimate load (P_{exp}/A_n) to the ultimate tensile strength (f_u) of the material. The ratio is called as the net section efficiency which represents reduction in load carrying capacity. The comparisons between the test results and the predicted values computed according to codal provisions have been discussed in the previous sections. Based on the above comparisons, geometrical factors such as connection eccentricity ($\bar{x}$), connection length (L), width of connected leg of the angle (a_c), net width of connected leg of the angle (a_{cn}), width of unconnected leg (a_d), nominal bolt diameter (d) and angle thickness (t) have effect on net section efficiency. Therefore, new net section efficiency (U) equation is developed for both single and double angles incorporating the above geometrical factors. In order to establish the form of the equation, regression analysis including linear and non-linear regression analysis have been performed using commercially available statistical software Sigmaplot 10. The net section efficiency equation is

$$U=1.024-0.301(\bar{x}/L)-\left(0.12a_{cn}+0.22a_d-0.761d-1.5t\right)/a_c$$

Based on the net section efficiency equation, it is recommended that for cold-formed steel angle members, the nominal tensile strength (P_{un}) of angle sections can be calculated as

$P_{un} = UA_nf_u.$

where A_n = Net area of cross section

U = net section efficiency

CONCLUSIONS

Based on the experimental, theoretical and numerical investigations the following conclusions are made.

1) The ultimate load carrying capacity increases as the cross – sectional area increases. Provision of more number of bolts improves the connection rigidity which also contributes to increase in load carrying capacity.
2) The presence of lip increases the load carrying capacity of single angles by 22%. The load carrying capacity increases by 26% for double angles connected to the opposite side of the gusset than the connected to same side of gusset plate.
3) Cold-formed steel angles with larger outstanding legs experiences local buckling under eccentric tensile loading.
4) In case of single angles the values predicted by the international codes AISI and AS/NZS are nearly 11% lower than the experimental ultimate loads irrespective of whether the angle is equal or unequal and provided with or without lip.BS code underestimates the values for single angles by 29% with respect to experimental ultimate loads.

5) In case of double angles the ultimate loads predicted by the AISI and AS/NZS are nearly 20% lower than the experimental ultimate loads. BS code underestimates the values for double angle members by 21% with respect to experimental ultimate loads.
6) The stress contours obtained in the finite element analysis indicates that maximum stresses occur in the innermost bolt holes from which the experimental failures were initiated.
7) The proposed equation for net section efficiency is applicable only when longer leg of the angle is connected.

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