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# Study on Behaviour of Bolted Cold-formed Steel Angle Tension Members

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## Abstract

**Background:** The main objective is to investigate the behaviour of cold-formed steel single and double angles subjected to tension. **Methods:** Sixteen single plain and lipped angle specimens and thirty two numbers of double angle specimens connected to the same side and opposite side of the gusset plate were tested in a Universal Testing Machine using ordinary black bolts of 10mm diameter. The experimental loads are compared with ultimate load calculated using BS 5950(Part V)-1998, and AS/NZS 4600:2005. **Results:** Most of the single and double angles failed by net section fracture failure and the outstanding leg are subjected to local bend due to shear lag effect. **Application:** Angles are used in a variety of structures such as trusses, transmission towers etc. as tension members. Angles may be used as single angles or double angles and the connection may be bolted or welded.

**Keywords:** Cold-Formed Single and Double Angles, Shear Lag, Tension Members

## 1. 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. The reduction in load carrying capacity occurs due to a phenomenon as shear lag effect. The study of shear lag effect on single and double angles made of hot rolled sections which included different cross-sectional configurations, connection materials and fabrication methods were based on test results of 218 specimens[1]. To investigate the effect of shear lag tests on cold formed steel channel sections with different dimensions were conducted and the test results were compared with predictions computed based on several specifications[2]. The material nonlinear effects of 20 node quadratic brick element were modeled by assuming the material stress-strain curve to be elastic perfectly plastic[3]. The effect of shear lag on the net section rupture capacity were conducted by comparing experimental and finite element results of 24 specimens which includes single and double angle tension members[4].

The resulting stress distribution justified the block shear strength equation by use of area along the gross shear plane. The von Mises stresses indicate that block shear failure might occur in a two bolt connection, and net section failure might occur in three and four bolts connection[5]. The factor of safety for angles under tension in the limit state format giving due considerations to block shear failure and yielding of gross section was obtained[6]. The knowledge and understanding of the behaviour of cold-formed steel bolted connections to determine tensile capacity, bearing capacity and the interaction of tension and bearing capacities were performed[7]. An equation for shear lag factor which represents the net section reduction coefficient has been suggested[8]. An expression for net section efficiency (U) which depended on the geometrical factors such as connection eccentricity ($\bar{x}$), 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

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thickness (t) has been suggested[9]. The vulnerability of an official 10-story steel moment resistant frame, designed according to Iranian National Building Codes (INBC) is assessed[10].

All the above investigations were made for the hot rolled 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 subjected to tension.

## 2. 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.

**Australian/NewZealand Standards: AS/NZS 4600-2005**[11]

The nominal section capacity of a member in tension $N_{t,}$ shall be taken as the lesser of

$$N_t = A_g f_y \text{ and} \tag{1}$$

$$Nt = 0.85\, K_t A_n f_u \tag{2}$$

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**[12]

The tensile capacity $P_t$, of a member

$$P_t = A_e{}^{*} p_y \tag{3}$$

**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.

## 3. Experimental Investigation

Sixteen single plain and lipped angle specimens and thirty two numbers of double angle specimens connected to the same side and opposite side of the gusset plate were tested in a Universal Testing Machine using ordinary black bolts of 10mm diameter. Standard tension tests were conducted on coupons and the modulus of elasticity was found to be $2x10^5 N/mm^2$. The yield stress of 2mm and 3mm thickness was 210 and 228 $N/mm^2$. Similarly the ultimate stress of 2mm thickness was $268N/mm^2$ and 3mm thickness was 292 $N/mm^2$. The thickness of mild steel gusset plate used was 8mm at both the ends. The length of the specimen was 500mm. For single angle specimens gusset plates were not reused and for double angle specimens they were reused. The angles were connected to gusset plate by larger leg. Dial gauge was used for measuring elongation. The edge distance and pitch distance were adopted as per Indian standards.

Figure 1 and 2 present the details of the fabricated single and double angle specimens. During the loading process the failure pattern was recorded. Figure 3, 4, 5 shows the experimental setup for all single, double angle specimens tested and Figure 6 shows the gusset plates and bolts used for the connection.

## 4. Numerical Investigation

This investigation analysis aims to develop a model that could study the behaviour of bolted cold-formed steel single and double angle tension members. SHELL 63 element type was used to model the single and double angle specimens. 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

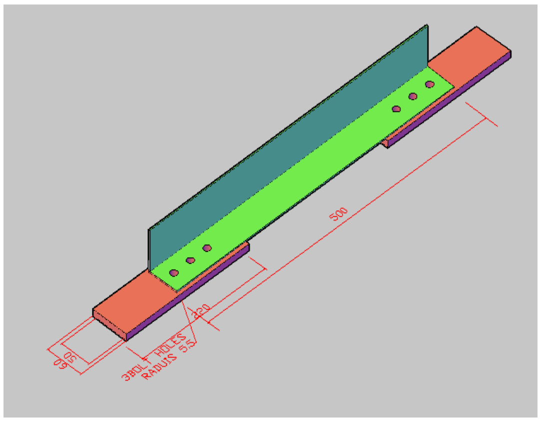


**Figure 1.** Single lipped angle specimen connected to gusset plate (t = 3mm).

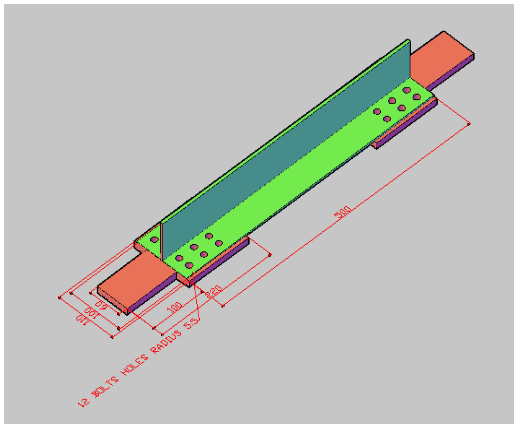


**Figure 2.** Double angle specimen connected to opposite side of the gusset plate (t = 2mm).

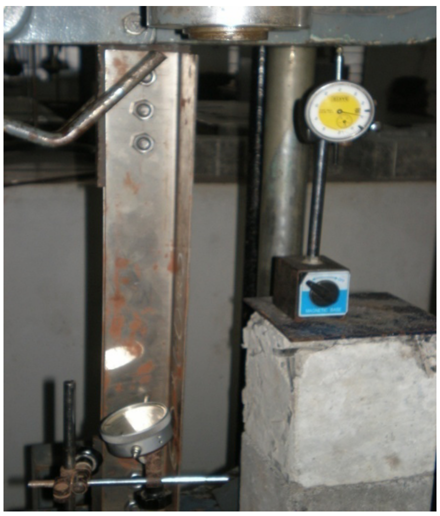

**Figure 3.** Experimental setup for single plain angle specimen (t = 2mm).

**Figure 4.** Experimental setup for double angle specimen connected to same side of the gusset plate (t = 3mm).

**Figure 5.** Experimental setup for double plain angle specimen connected to opposite side of the gusset plate (t = 2mm).

**Figure 6.** Gusset plates and bolts used for connections.

mesh of the model is shown in Figure 7 and 8. 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.

## 5. Result and Discussion

The behaviour of cold-formed steel single and double angles when subjected to eccentric tension were studied. The experimental ultimate load carrying capacity of the specimens tested were compared with the load carrying capacities predicted using Australian/New Zealand and British standards. The experimental results were also compared with the numerical results obtained using ANSYS software.

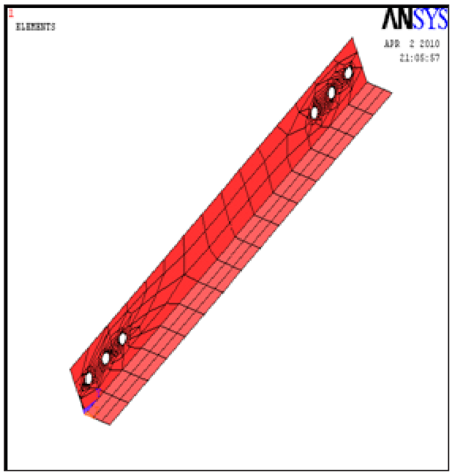


**Figure 7.** Element meshing for single lipped angle 50x50x2.

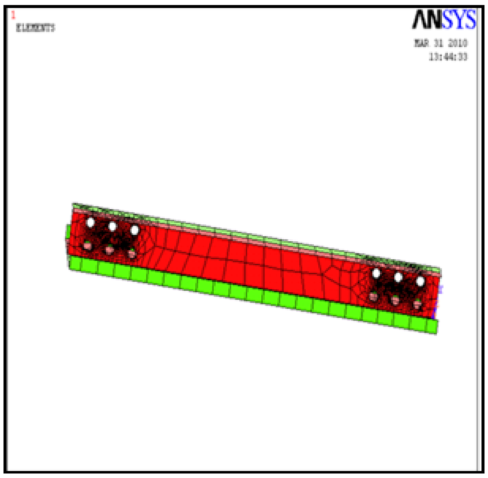


**Figure 8.** Element meshing for double angle connected to same side of the gusset plate (50x50x15x3).

### 5.1 Experimental Investigation

#### *5.1.1 Ultimate Load-carrying Capacity*

The experimental ultimate loads for all the cold-formed steel single and double angles are presented in Table 1, 2 and 3. There is increase in load carrying capacity by 10% for lipped angles when compared to plain angles. Similarly, in case of double angles the load carrying capacity increases by 8% for angles connected to opposite side of gusset plate than to the same side of the gusset plate.

#### *5.1.2 Load vs Deflection*

Figure 9 and 10 show the typical load versus deflection behaviour for single angles with and without lips and

**Table 1.** Ultimate load carrying capacity of the single angles

| S. No. | Size of the specimen (mm) | Ultimate load carrying capacity in kN t = 2mm | | Ultimate load carrying capacity in kN t = 3mm | |
|---|---|---|---|---|---|
| | | Single line | Double line | Single line | Double line |
| 1 | 50x50xt | 19 | 20 | 50 | 51 |
| 2 | 60x60xt | 20 | 21 | 52 | 54 |
| 3 | 50x50x15xt | 23 | 24 | 53 | 55 |
| 4 | 60x60x15xt | 24 | 26 | 55 | 57 |

**Table 2.** Ultimate load carrying capacity of the double angles

| S.No | Size of the specimen (mm) | Ultimate load carrying capacity in kN t = 2mm | |
|---|---|---|---|
| | | Single line | Double line |
| **Double angles connected to opposite side of gusset plate** | | | |
| 1 | 50x50xt | 35 | 40 |
| 2 | 60x60xt | 41 | 42 |
| 3 | 50x50x15xt | 46 | 47 |
| 4 | 60x60x15xt | 51 | 56 |
| **Double angles connected to same side of gusset plate** | | | |
| 1 | 50x50xt | 33 | 35 |
| 2 | 60x60xt | 39 | 40 |
| 3 | 50x50x15xt | 45 | 45 |
| 4 | 60x60x15xt | 49 | 55 |

**Table 3.** Ultimate load carrying capacity of the double angles

| S.No | Size of the specimen (mm) | Ultimate load carrying capacity in kN t = 3mm | |
|---|---|---|---|
| | | Single line | Double line |
| Double angles connected to opposite side of gusset plate | | | |
| 1 | 50x50xt | 108.7 | 109 |
| 2 | 60x60xt | 112.8 | 114 |
| 3 | 50x50x15xt | 116 | 118 |
| 4 | 60x60x15xt | 119 | 121 |
| Double angles connected to same side of gusset plate | | | |
| 1 | 50x50xt | 105 | 107 |
| 2 | 60x60xt | 110 | 111 |
| 3 | 50x50x15xt | 112 | 115 |
| 4 | 60x60x15xt | 115 | 119 |

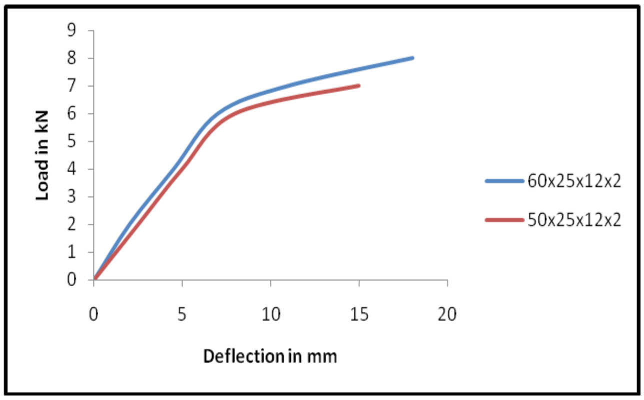


**Figure 9.** Load vs Deflection behaviour of single lipped angle specimen(t = 2mm).

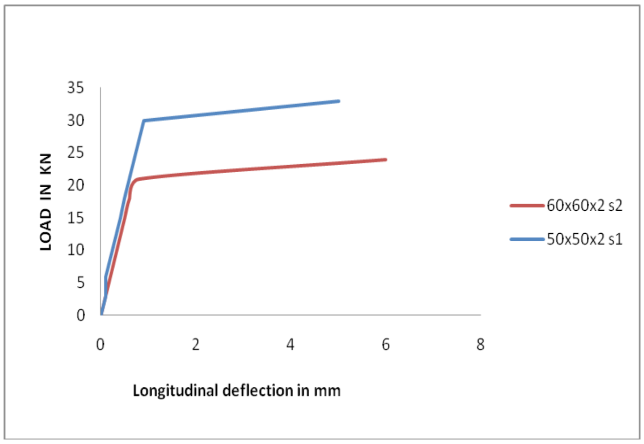


**Figure 10.** Load vs Deflection behaviour of double plain angle specimen connected to opposite side of the gusset plate(t = 3mm).

double angles. From the graphs, the ultimate load carrying capacity increases as the cross-sectional area in the connection increases. The stiffness of the member increases when the rigidity of the connection increased.

### *5.1.3 Modes of Failure*

Failure pattern for all single and double angles were noticed during testing. Block shears failure and net section fracture failure was observed and is given in Table 4, 5 and 6 for all the single and double angles tested. Figure 11 and 12 shows the block shear failure and net section fracture failure of the specimens tested. It depends upon the cross section and rigidity of connection.

There was a gap between the corner of the connected leg and the gusset plate. The visible length of gap was usually from the edge of the angle to the innermost bolt and was generally 10mm. Generally larger gaps were associated with greater eccentricity of the cross-section, smaller angle thicknesses and shorter connection lengths.

No major slips of the connections were observed during the tests. All the specimens failed at the inner most bolt hole as the ultimate load was reached. After necking, the critical cross-section was torn out from the edge of the

**Table 4.** Mode of failure of the single angles

| S. No. | Size of the specimen (mm) | t = 2mm | | t = 3mm | |
|---|---|---|---|---|---|
| | | Single line | Double line | Single line | Double line |
| 1 | 50x50xt | Net section | Net section | Net section | Net section |
| 2 | 60x60xt | Net section | Net section | Block shear | Net section |
| 3 | 50x50x15xt | Net section | Net section | Net section | Net section |
| 4 | 60x60x15xt | Net section | Net section | Net section | Block shear |

**Table 5.** Mode of failure of the double angles

| S.No | Size of the specimen (mm) | t = 2mm | |
|---|---|---|---|
| | | Single line | Double line |
| Double angles connected to opposite side of gusset plate | | | |
| 1 | 50x50xt | Net section | Net section |
| 2 | 60x60xt | Net section | Net section |
| 3 | 50x50x15xt | Block shear | Block shear |
| 4 | 60x60x15xt | Net section | Block shear |
| Double angles connected to same side of gusset plate | | | |
| 1 | 50x50xt | Net section | Net section |
| 2 | 60x60xt | Net section | Block shear |
| 3 | 50x50x15xt | Block shear | Net section |
| 4 | 60x60x15xt | Block shear | Net section |

**Table 6.** Mode of failure of the double angles

| S.No | Size of the specimen (mm) | t = 3mm | |
|---|---|---|---|
| | | Single line | Double line |
| Double angles connected to opposite side of gusset plate | | | |
| 1 | 50x50xt | Net section | Net section |
| 2 | 60x60xt | Block shear | Block shear |
| 3 | 50x50x15xt | Net section | Block shear |
| 4 | 60x60x15xt | Net section | Net section |
| Double angles connected to same side of gusset plate | | | |
| 1 | 50x50xt | Net section | Net section |
| 2 | 60x60xt | Net section | Net section |
| 3 | 50x50x15xt | Block shear | Net section |
| 4 | 60x60x15xt | Net section | Block shear |

**Figure 11.** Mode of failure for single plain angle specimen (Block shear failure) (t = 3mm).

**Figure 12.** Mode of failure for double plain angle specimen connected to opposite side of gusset plate (Net section failure) (t = 2mm).

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. The outstanding leg is subjected to local bend due to shear lag effect. Failure load of all the single and double angles tested are presented in Table 7, 8 and 9.

## 5.2 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 AS/NZS:4600-2005, BS:5950 (Part 5 )-1998 is made.

The comparison of predicted ultimate loads by the various codes for single and double angles tested is shown in Figure 13 and 14. The tensile capacity equation of the

**Table 7.** Failure load of the single angles

| S. No. | Size of the specimen (mm) | Failure load in kN t = 2mm | | Failure load in kN t = 3mm | |
|---|---|---|---|---|---|
| | | Single line | Double line | Single line | Double line |
| 1 | 50x50xt | 8 | 8 | 30 | 32 |
| 2 | 60x60xt | 7.5 | 7.5 | 10 | 13 |
| 3 | 50x50x15xt | 13 | 8 | 33 | 30 |
| 4 | 60x60x15xt | 10 | 13 | 11 | 16 |

**Table 8.** Failure load of the double angles

| S.No | Size of the specimen (mm) | Failure load in kN t = 2mm | |
|---|---|---|---|
| | | Single line | Double line |
| Double angles connected to opposite side of gusset plate | | | |
| 1 | 50x50xt | 11 | 20 |
| 2 | 60x60xt | 12 | 25 |
| 3 | 50x50x15xt | 19 | 14.6 |
| 4 | 60x60x15xt | 40 | 24 |
| Double angles connected to same side of gusset plate | | | |
| 1 | 50x50xt | 10 | 10 |
| 2 | 60x60xt | 25 | 15 |
| 3 | 50x50x15xt | 10 | 20 |
| 4 | 60x60x15xt | 20 | 15 |

**Table 9.** Failure load of the double angles

| S.No | Size of the specimen (mm) | Failure load in kN t = 3mm | |
|---|---|---|---|
| | | Single line | Double line |
| Double angles connected to opposite side of gusset plate | | | |
| 1 | 50x50xt | 24.2 | 26 |
| 2 | 60x60xt | 65.2 | 55 |
| 3 | 50x50x15xt | 17.8 | 40 |
| 4 | 60x60x15xt | 100.5 | 80 |
| Double angles connected to same side of gusset plate | | | |
| 1 | 50x50xt | 21.5 | 20 |
| 2 | 60x60xt | 31 | 36 |
| 3 | 50x50x15xt | 7.3 | 10 |
| 4 | 60x60x15xt | 31.5 | 39 |

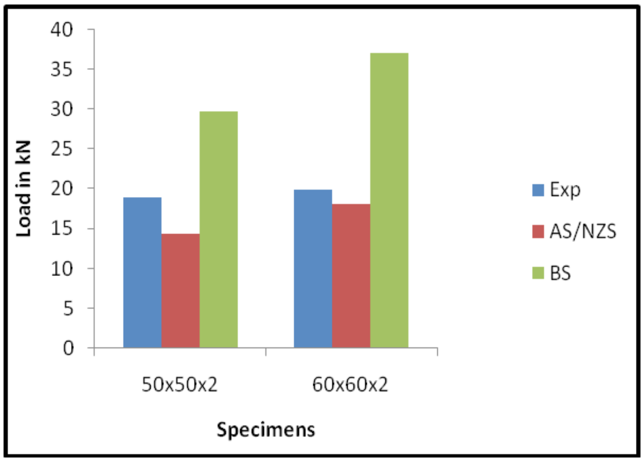


**Figure 13.** Comparison of ultimate loads with loads based on codal provisions for single plain angles (t =2mm).

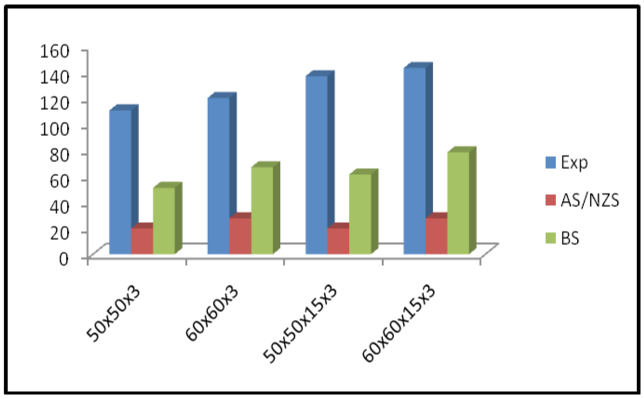


**Figure 14.** Comparison of ultimate loads with loads based on codal provisions for double angles connected to same side of the gusset plate (t = 3mm).

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. The values predicted by BS are higher than the experimental loads and loads calculated by AS/NZS for single angles. For double angles the values predicted by AS/NZS and BS are lesser than experimental values.

## 5.3 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. ANSYS employs Newton-Raphson equilibrium iterations. The general post processor in ANSYS is used to review results at each load increments. Figure 15 and 16 shows the stress distribution of the single and double angles upto $0.87f_y$.

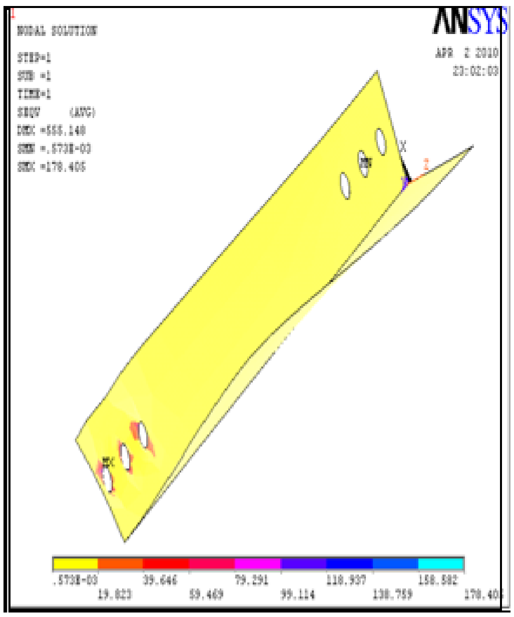


**Figure 15.** Stress distribution for single plain angle 50x50x2

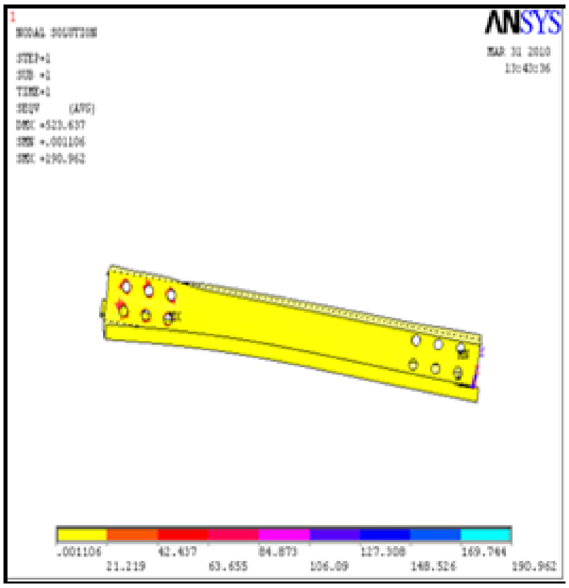


**Figure 16.** Stress for double lipped angle connected to same side of the gusset plate (50x50x15x3).

## 6. Summary and Conclusion

This study investigates the behaviour of cold-formed steel single and double angles subjected to tension. The results obtained in this research are summarized as follows.

- When the cross sectional area increases the ultimate load carrying capacity increases. The connection rigidity increases when number of bolts are more
- There was an increase in load carrying capacity increases by 10% for lipped angles when compared to plain angles. Similarly for the increase is 8% for double angles connected to the opposite side of the gusset than the connected to same side of gusset plate
- The outstanding leg is subjected to local bend due to shear lag effect
- Most of the single and double angles failed by net section fracture failure
- In case of single angles the values predicted by BS are higher than the experimental loads. In case of double angles the experimental loads are higher than AS/NZS and BS.
- 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.
- The stress distribution obtained in the numerical investigation closely agrees with the experimental results.

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