Connection Failure of Light weight Gauge Steel Angle Sections Subjected To Tensile Loading Conditions

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ABSTRACT
Tests conducted in cold formed steel angles are presented, connected by bolts and submitted to tensile loads. When the bolted connection is considered, the angle does not deform evenly, showing the phenomenon known as shear lags. This coefficient reduces the net section and reduces the resistant capacity of the tensioned steel angle. A tension member was loaded until strain hardening is reached and elongates excessively before the fracture causing the members to fail at the angle. This Research paper deals with the failure modes for single and double- angle specimens were observed during the test. The experimental work was conducted on a universal testing machine. In the experimental investigation 36 specimens were carried out on tension members fastened with bolts, It concluded that the stiffness of the connection increases also the stiffness of the member.

Keyword: *EBC, Block shear, AISC-LRFD, Mode of failure, Buckling, Angle sections, CFS*

I. INTRODUCTION
The design of industrial building is governed mainly by functional requirements and the need for economy of construction. Gradually, as industrial processes progressed, various steel products became available of rolled members and cold-formed elements. The metallurgical process of Hot rolling, used mainly to produce sheet metal or simple cross sections from billets describes the method of when industrial metal is passed or deformed between a set of work rolls and the temperature of the metal is generally above its recrytallization temperature. Cold Rolled Steel rolling processes at temperatures that are close to normal room temperature are used to create cold rolled steel. This increases the strength of the finished product through the use of strain hardening by as much as 20 percent.

Although cold-formed steel is used for several products in building construction, framing products are different in that they are typically used for wall studs, floor joists, rafters, and truss members. Examples of cold-formed steel that would not be considered framing includes metal roofing, roof and floor deck, composite deck, metal siding, and purlins and girts on metal buildings.

II. LITERATURE REVIEW
Bouchair in 2008 studied analysis of the behavior of stainless steel bolted connections and conducted experiments on two types of bolted connections that were common in steel structures. They concerned cover plate connections and T-stubs, where the bolts were loaded in shear or in tension. The requirements for stainless steel connection design were essentially the same as for carbon steel. The study considered the case of austenitic stainless steel for which the conventional elastic limit was relatively low compared to the ultimate strength. In bearing, criteria on deformation limits have to be considered for cover plate connections. In T-stubs, strain hardening of stainless steel exhibits a continuous increase of the applied load and can influence the failure mode. A Finite Element Model is developed and validated for the two types of connections. A more extensive parametric study should be carried out to develop a better understanding of the behavior of stainless steel connections.

Gotluru (2000) studied the behavior of cold formed steel beams having open sections, which were subjected to torsion. They focused only on beams subjected to bending and torsion. They conducted a series of experimental study on C-sections and compared the results with simple geometric analysis, finite element analysis and finite strip analysis results. They observed that when the unbraced thin walled beams were subjected to transverse loads applied away from the shear centre, lateral buckling of beams did not occur suddenly but took place by displacing horizontally and by rotating gradually. They also reported that the serviceability limit state might be more critical than the strength limit state and that warping restraint at the ends depends on type of supports. They presented an expression to find partial restraint based on support condition in terms of warping spring stiffness. They also found that local buckling load decreased with the rotation of the beam and the lateral torsional buckling load increased.

Yu & Schafer (2003) studied the bending behaviour of C and Z sections to present an accurate prediction for the common sections in use. The details of the specimen were so chosen such that local buckling was free to form by restricting distortional buckling and lateral torsional buckling. They conducted a series of local buckling tests on C and Z beams and presented the effect of web slenderness on the local buckling failures of C and Z sections. They had also highlighted the discontinuities and inconsistencies in the American Iron and Steel Institute (AISI), Canadian Standards Association (S136) design provisions for stiffened web elements under stress gradient. They concluded that the Direct Strength Method provided the best test-to-predicted ratios for both slender and stub specimens and demonstrated that many improvements in the elastic buckling and effective width calculations were possible for C and Z sections.

Hancock (2003) presented a review article on Cold formed Steel structures due to enhanced development in the field of cold formed steel applications. He consolidated and summarized the major research developments in cold formed steel for three years (from 1999 to 2001), the development of North American Specifications for the design of Cold formed Steel Structures and finally presented a brief summary of Direct Strength method that was developed by American Iron and Steel Institute Specifications Committee.

Kulak and Grondin (2001) performed a statistical study on evaluation of LRFD rules for block shear capacities in bolted connections with test results. It was stated that there were two equations to predict the block shear capacity but the one including the shear ultimate strength in combination with the tensile yield strength seemed unlikely. Examination of the test results on gusset plates reveals that there is not sufficient tensile ductility to permit shear fracture to occur.

III. BLOCK SHEAR STRENTH

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. 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. The knowledge and understanding of the behavior ofcold-formed steel bolted connections to determine tensile apacity, bearing capacity and the interaction of tension and bearing capacities were performed. The design of industrial building is governed mainly by functional requirements and the need for economy of construction. Gradually, as industrial processes progressed, various steel products became available of rolled members and cold-formed elements. The metallurgical process of Hot rolling, used mainly to produce sheet metal or simple cross sections from billets describes the method of when industrial metal is passed or deformed between a set of work rolls and the temperature of the metal is generally above its recrytallization temperature. Cold Rolled Steel rolling processes at temperatures that are close to normal room temperature are used to create cold rolled steel. This increases the strength of the finished product through the use of strain hardening by as much as 20 percent.

Although cold-formed steel is used for several products in building construction, framing products are different in that they are typically used for wall studs, floor joists, rafters, and truss members. Examples of cold-formed steel that would not be considered framing includes metal roofing, roof and floor deck, composite deck, metal siding, and purlins and girts on metal buildings.

IV. ETHIOPIAN BUILDING CODE STANDARD EBCS 3, 1995

According to the EBCS 3 Specification, axially loaded tension members designed to resist a factored axial force of $N_{t,Sd}$, calculated using appropriate load combinations, must satisfy the condition:

$$N_{t.sd} \leq N_{L,RD}$$

where:

$N_{t,\ Rd}$ = design tension resistance capacity of the cross-section, taken as a smaller of either the design plastic resistance $N_{pl,RD}$ of the gross section or the design ultimate resistance Nu,Rd of the net section at the bolt hole where, again, $N_{pl,Rd}$ and $N_{u.Rd}$ are determined as in the fol1awing expressions:

$$N_{Pi,RD} = \frac{A_g\ x\ f_y}{\gamma_{MO}}$$

$$N_{U,\ RD} = \frac{0.9\,x\,A_{eff}\ x\ f_U}{\gamma_{M2}}$$

The partial safety factor γ_{MO} = 1.1 and while γ_{M2} = 1.25 represents resistance of the net section at bolt holes.

V. AISC-LRFD SPECIFICATION

According to the AISC-LRFD Specification, tension members designed to resist a factored axial force of P_u, calculated using the appropriate load combinations, and must satisfy the condition:

$\phi_t P_n \geq P_u$

Where:

$ØP_n$ = the design tensile strength of the cross section and it is evaluated based on three limit states: yielding in gross section, fracture in effective net section, and block shear. $Ø_t$ = 0.9 is the appropriate resistance factor in tension.

Yielding in the cross section away from the joint should be avoided to prevent excessive deformation that results when steel yields. The design strength for this limit state is evaluated from the equation:

$Ø_t P_n = Ø_t \ x\ f_y \ x\ A_g$

where

$Ø_t$ = 0.9 = resistance factor for tension

f_y = specified minimum yield stress of steel

P_n = nominal axial strength

Fracture in effective net section or fracture of the net section the joint should be avoided, to prevent the loss of load-carrying capacity of the member. The design strength for this limit state is evaluated from the equation:

$Ø_t P_n \geq Ø_t \ x\ f_u \ x\ A_e$

where:

Øt = 0.75 = resistance factor for fracture in tension

F_u = specified minimum tensile strength of the material

A_e= effective net cross-sectional area of the member

P_n = nominal axial strength

For members without holes, fu1ly connected by welds, both A_{eff} in EBCS 3 and A_e in AISC-LRDF specifications are the smal1er of the gross area of the member and the effective area of the welds. As it can be seen from both EBCS 3 and AISC-LRDF specifications, the concept of net section forms one of the criteria for the determination of limiting strength of the cross section.

VI. NET SHEAR AREA FOR BLOCK SHEAR.

The design value of the effective resistance $V_{eff,Rd}$ for rupture along a block shear failure path shall be determined from:

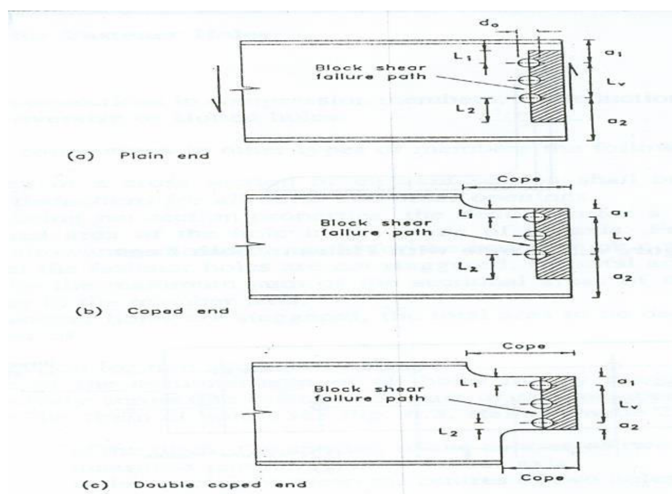

Figure 1 Net Shear Area for Block Shear

$$V_{eff.RD} = \frac{0.6x\,f_y\ x\ A_{v.ef}f}{\gamma_{MO}}$$

Where γ_{MO} = 1.1 = partial safety factor

fγ = specified minimum yield stress of steel

$A_{v,eff}$ = effective shear area subject to block shearThe effective shear area $A_{v,eff}$ for block shear, Fig. 4.2 is determined from:

$A_{v,eff} = t (Lv + L1 + L2 – nd_o)$

in which L_1 and L_2 are given by:

$L_1 = 5.0d_0 \leq a_1$

$L_1 = 2.5d_0 \leq a_2$

and n = the number of fastener holes in the block shear failure path

d_o = hole diameter

T = thickness of the web or bracket

VII. 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. The failure modes are different for single and double angle sections. The mode of failure depends upon the cross section and rigidity of connection. The different mode failures observed during the tests are discussed here.

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. buckling of angle specimen

Figure 2 Local Buckling of Single Angles

The mode of failure of all the specimens tested with three different end conditions 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 conditions are discussed.

7.1 Local plate buckling

Local plate buckling was observed in the case of single plain angles tested with ball end condition. 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. Similarly, the lipped single angles tested as stub columns with ball end condition failed by local buckling. The local buckling in the case of stub columns occurred at mid height of the section either in the lip or in the flange. Figure 6.42 shows the tested specimens of single angles with ball end condition. In the case of lipped double angles welded back-to-back the failure was initiated by local plate buckling for short columns. The failure was between the mid height of the section and one-third height of the section caused either in the lips or in the flanges. Figure 8.15 shows the failure of single angle by local plate buckling tested with bolted end condition.

7.2 Flexural buckling

Lipped single angles tested as short columns with ball end condition failed by overall flexural buckling. Failure of plain 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. Similarly, single short columns with lip under welded end condition failed by flexural buckling. Figure shows the tested specimens of double angles welded back-to-back with bolted end condition. Plain single angles tested as short columns with bolted end condition failed by flexural buckling at one-fourth height of the section or by local plate buckling initiated at the end of the section. Similarly, lipped single angles tested as short columns with bolted end condition failed either by flexural or torsional flexural buckling.

7.3 Distorsional buckling

Lipped double stub angles welded back-to-back with ball end condition failed by torsional-flexural buckling. Figure 8.17 shows the tested specimens of double angles welded back-to-back with ball end condition. Torsional-flexural buckling was noticed in all the double angles except in lipped short columns when tested with welded end condition. Figure 6.46 shows the failure of starred angle by torsional-flexural buckling. Lipped short columns of double angles with bolted end condition also failed by torsional- flexural buckling.

Fig 3 Distortional buckling

7.4 Torsional buckling

It is observed that failure is caused either by flexural or torsional buckling in the case of starred plain angles tested with ball end condition.

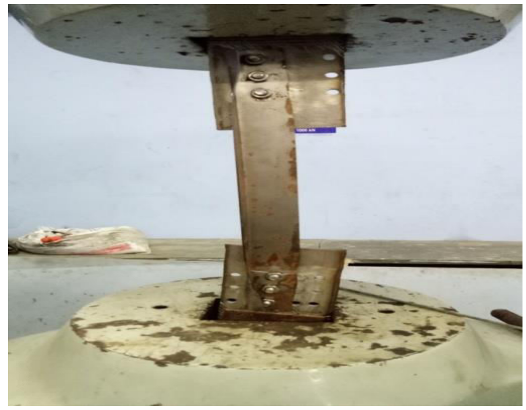

Figure 4 Tearing failure of Unequal angle Block shear failure

Because of simultaneous yielding of two innermost bolt holes, the net section area reduces considerably, which results in block shear failure. This type of failure occurs in a faster rate compared to tearing failure. Figures 8.20 shows the block shear failure of all single angle specimens with and without lips and double angles connected to opposite side and same side of the gusset plate respectively.

Figure 5 Block shear failure

7.5 Net section fracture failure

The failure is initiated at the inner most bolt hole and the crack propagates towards the edges in a direction perpendicular to the direction of loading. Figures shows the net section failure of all single angle specimens with and without lips and double angles connected to opposite side and to same side of the gusset plate respectively. It is also observed that when the connected leg fails by net section fracture, the outstanding leg which is subjected to compressive stress suffers local buckling.

7.6 Partial Rupture of the Net Section

The typical specimen failure consisted of a partial rupturing of the net section followed by the tearing of the web. The peak load was reached before fracturing of the full net section, but after rupture of the partial net section. Fracturing of a specimen's web initiated at the lead bolt hole and propagated to the web's outside edge. Necking down of the tension plane area preceded fracture. Specimens with small eccentricities exhibited a significant amount of bolt hole deformation. However, it was observed that the amount of deformation decreased with increasing eccentricity. Those specimens with the largest eccentricities demonstrated very minor hole deformation. Fig.8.23 shows a typical partial net section failure.

Figure 6Partial Net Section Rupture

VIII. CONCLUSION

A tension member was loaded until strain hardening is reached and elongates excessively before the fracture causing the members to fail at the angle. The failure modes for single and double- angle specimens were observed during the test. global bending, local bending, local buckling, tearing failure, shear failure, net fracture fracture. showed that the ultimate load carrying capacity increases with the cross- sectional area and the number of bolts in the connection. It is also observed that the stiffness of the connection increases also the stiffness of the member.

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