A REVIEW PAPER ON FORMULATION OF BLOCK SHEAR FAILURE IN STAGGERED BOLT CONNECTION UNDER TENSION

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Abstract : Block shear failure and strength in bolted connection is complex function mode in steel connection because many uncertainties occurs between yielding strength, rupture strength, net area, gross area etc. The aim of this review paper to develop a new numerical or equational function that covers all parameter regarding block shear strength and its failure.

***Keybroad* - Block shear Strength, Block shear failure, Finite element method, Staggered bolt connection, shear rupture, Tension member, Tension rupture.**

I. INTRODUCTION

Block shear is a potential failure mode of member subjected to combination of axial tension. In this failure, a block of member is partially or entirely removed from the parent member along the fastening of bolt connection arrangement. Many design provision are predicted against block shear failure of bolted connections since its discovery in 1978 by Bikemoe and Gilmore. Most important factor of block shear is the presence of varying stress state conditions that cause failure on tension and shear plane.

Block shear failure is possible when the material bearing and bolt shear strengths are higher. The block shear capacity depends on the areas of tensile and shear stress. According the IS: 800: 2000, shear and tensile planes rum parallel and perpendicular to the axial load direction. Block shear strength Tdb of the connection is taken as following equation.

$$\left.\begin{array}{l} T_{db} = \dfrac{Avg\,.fy}{\sqrt{3}\,\gamma mo} + 0.9\,\dfrac{Atn\,.fu}{\gamma mi} \\[2ex] \qquad\text{OR} \\[2ex] T_{db} = 0.9\,\dfrac{Avn\,.fu}{\sqrt{3}\,\gamma mi} + 0.9\,\dfrac{Atg\,.fy}{\gamma mo} \end{array}\right\}\ \text{take } T_{db} \text{ minimum as block shear strength}$$

This current block shear strength equation is not define for staggered bolt connection arrangement, due to uncertainty of gross and net areas in computing yielding and rupture resistance terms. It is very necessary to find out the equation for block shear strength in staggered bolt arrangement because mostly steel connection of bolt are in diamond arrangement which is staggered arrangement, for its better efficiency. So it is needed to predict the equation for block shear capacity.

II. METHODOLOGY

The all literature searched from authentic journals and confesses from the online library data base of Integral University Lucknow, Science direct, Taylor & Francis, Elsevier, Research Online, Missouri S&T, Asian Journal of Civil Engineering and other various relevant sources. In starting, generally web searched is done by using some common keywords like “Block Shear”, “staggered bolts”,” FEM”, “tension member”, “bolts connection”, “cold-formed steel”, “block shear failure”, “stress distribution”, “shear rupture”, “failure criteria”, “fracture path”, “fracture profile” etc.
After detailed search on publication regarding block shear failure and strength in staggered bolt connection under tension, the selection criteria of paper is identify to meet the topic or purpose of this paper.

III. LITERATURE REVIEW

G. L. Kulak *et al.* (1983) [1] had tested full scale of double-row bolted-web connections were performed on coped and non-coped ASTM grade A36 W460X89 specimens. The major variables were end and edge distances, slot length, and number of holes. 3/4-

A325 (19mm) bolts and a hole diameter of 21 mm were used in the connections. The minimum edge and end distances were 25 mm. they concluded that indicated that shear resistance is developed on the gross section rather than the net section.

B. B. . Huns *et al.* (1985) [2] tested 28 specimens to develop an improved design method for gusset plates. Gage between lines of bolts, edge distance, bolt spacing and number of bolts were considered as the strength parameters. Gusset plates fastened with two lines of bolts were tested. Test specimens had a gage length of 51, 76 and 101 mm, edge distance of 25, 38 mm, and pitch distance of 38 and 51mm. Connections had two to five bolts in a bolt line and diameter of bolt holes were 14 and 17 mm. The average material properties of 27 specimens had yield strength of 229 MPa and an ultimate strength of 323 MPa. One specimen had a yield strength value of 341 MPa and ultimate strength of 444 MPa. Test plates had a basic failure mode consisting of tensile failure across the last row of bolts, along with an elongation of the bolt holes. Load deformation curves of the each test specimens was obtained and it was observed that the drop in strength from the ultimate load to second strength plateau corresponded approximately to the ultimate strength of the net area at the last row of bolts. Ultimate shear resistance was more difficult to define, because, the shear stress behavior varied among the test specimens. Shear stress was found to be dependent on the connection length and a new block shear capacity equation, which includes the connection length factor, was developed.

H. I. Epstein *et al.* (1991) [3] they investigated, using finite element analysis, the effect that the stagger of bolt patterns has on ultimate block shear capacity. Significant differences from current code practices were found. In particular, it was found that stagger may not always increase the tension capacity as codes indicated. Finite element analysis was made. The reduction in capacity due to negative stagger has significant design implications. Since design codes assume an increase in capacity when stagger (in either direction) is present, there may be a need to rethink these long-standing practices. The results indicate that a sign should be assigned to the stagger correction factor when stagger is present in combination with shear lag. Whether or not this same phenomenon was present, in the absence of shear lag, still needs to be investigated. Of course, any changes in code equations should probably be justified by experimentation. However, there was significant difficulty in verifying the phenomenon anticipated by these analytical results because the change in capacity due to a stagger was relatively small. Therefore, there may not be a great impetus to perform such tests. On the other hand, the design codes were based solely on geometric considerations. Therefore, they suggested additional analytical work together with some statistically meaningful experimentation that could help designers optimize connection geometries, and may influence the framers of future codes.

Epstein *et al.* (1992) [4] performed an experimental study on double-row, staggered, and non-staggered bolted connections of structural steel angles. The basic connections to be tested were pairs of angles, 8 mm thick, connected by two rows of 8 mm diameter bolts in two rows on a 150 mm leg. Outstanding legs of the angles vary between 90, 210 and 150 mm. An end and edge distances of 38 mm, a bolt diameter of 19 mm were used in the connections. The effect of several parameters in the connection geometry was investigated. Test results were compared with the current code provisions and a revised treatment was suggested by inclusion of a shear lag factor to the equation.

T.J. Cunningham *et al.* (1994) [5] says that block shear failure can control the load capacity of several different types of connections, including shear connections at the ends of coped beams, tension member connections and gusset plates. In all cases, failure developed as yielding and/or rupture in many geometry planes bounded the connection, so Cunningham provide additional factor influencing block shear behavior which depend on geometry and material parameter of member. The factors are tension planes gross area A_{sg}, shear plane gross area $A_{vg,}$ tension plane net area A_{tn}, shear plane net area A_{vn}, tensile yield strength F_y, ultimate tensile strength $F_{u.}$ Results numerical comparisons with the AISC LRFD and ASD.
Gross (1995) [6] tested ten A588 Grade 50 and three A36 steel single angle tension members with various leg sizes that failed in block shear. A588 Grade 50 steel had a yield and ultimate strength of 427 and 545 MPa and A36 steel had a yield and ultimate strength value of 310 and 469 MPa, respectively. Bolt holes having a diameter of 21 mm and a bolt hole spacing of 64 mm and an end distance of 38 mm was used in all specimens. The edge distance was varied between 32, 38, 44 and 50mm. Test results were compared with the AISC-ASD and AISC-LRFD equation predictions and it was observed that code treatments accurately predict failure loads for A36 and A588 specimens.

Epstein *et al.* (1996) [7] studied the effects of stagger and shear lag on the failure load of angles in this study. Angles were modeled with 20 node brick elements and elastic-perfectly plastic stress strain curve for steel was used in this analysis. A strain based criterion was used to determine the failure load of the member. The non-dimensional finite element results were compared with the full scale testing results.

Kulak *et al.* (1997) [8] observed the shear lag effect on the net section rupture of the single and double angle tension members. For practical reasons it is unusual to be able to connect the all legs of the angle and the influence of only one of the connected leg to the tensile capacity of 24 the connection is termed as shear lag. ANSYS was used in the analysis and quadrilateral shell elements that can include plasticity were used to model the angles and elastic quadrilateral shell elements were used to define the gusset plates. Kulak and Wu included the material and geometric nonlinearities in the analysis. The failure load was considered as the load corresponding to the last converge load step. The failure loads obtained from finite element modeling were compared with the full sale testing.

Z. Marković *et al.* (1998) [9] says that about behavior associated with bearing and net section failure of bolted connections loaded in shear. Failure mode behavior was documented with the use of observations recorded during the testing of a total of 176 bolted connection specimens (Rogers and Hancock, 1997, 1998a,b). Bearing behavior includes piling of the sheet material in front of the bolts, as well as the material tearing associated with out-of-plane sheet distortion. Recommendations concerning the procedure used to identify the net section fracture and bearing failure modes were made. In addition, a detailed discussion of the test data used in the development of the current AS/NZS 4600 and AISI design equations for net section fracture at connections was completed.

Orbison (1999) [10] tested 12 specimens that failed in block shear. Three of these analyzed specimens were L6X4X5/16 tension members having varying edge distances of 50.8 mm, 63.5 mm and 76.2 mm. Nine of the specimens were WT7X11 tension members with two, three or four bolt end connections having varying edge distances of 63.5 mm, 76.2 mm. A490 bolts in bearing, 25.4 mm in diameter and snug-tight, were used for all specimen connections. A pitch distance of 76.2 mm and an end distance of 63.5 mm were used. Experimental failure loads were compared with code treatments. Recommendations were given based on the ultimate load and the strain variation along the tension plane that was measured during the experiments.

Kulak *et al.* (2001) [11] 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. After reviewing the test results, it was observed that failure modes seen in gusset plates and coped beams are significantly different and use of equations mentioned in Table1 gives conservative predictions for gusset plates but they are not satisfactory for the case of coped beams. In angles block shear capacity is predicted well by these equations. As a conclusion, Kulak and Grondin recommended different equations for predicting the block shear capacities for gusset plates and coped beams to use.

Driver *et al.* (2004) [12] Reliability methods were used to provide a simple, unified approach to design for block shear failure of gusset plates, angles, tees, and coped beams. The method provides both an adequate and a consistent level of safety and predicts capacities that compare well with test results over a broad array of connection configurations. Of particular importance, the proposed approach also reflected the failure modes that have been observed in tests, which is an aspect that is lacking in some current design equations.

Dr. Mohan Gupta *et al.* (2004) [13] examined the block shear capacity of steel single as well as double angles, for bolt holes in one or more rows, and with staggered and non-staggered holes. Only those specimens that follow all provisions regarding minimum pitch, edge and end distances were included in [10]this study. Angles composed of high strength steel are not included in this study. An improved approach, to compute the block gross shear area of specimens that had bolt holes staggered such that the end distance along the row of bolts towards the heel is relatively more, is suggested. This area, A^*_{gv}, as per this improved approach is termed here as effective block gross shear area and is somewhat less than the block gross shear area, A_{gv}, as per current practice. There is a considerable improvement in values of professional factors when the concept of effective block gross shear area is used in the computations. The following simple equation was found to give adequate results for single as well as double angles, for bolt holes in one or more rows, and with staggered and non-staggered holes.

$R_b = f_u.\ A_{nt} + f_{ys}.\ A^*_{gv}$

Bino B.S. Huns *et al.* (2004) [14] observed that failure modes in gusset plates located in tension consists of tearing of a block of material in a combination of tension rupture plus shear yield or rupture. Develop a finite element procedure to predict the behavior of tension and shear block failure from yielding of the gusset plates to rupture along the tension face and subsequent rupture along the shear faces. Two connection configuration were investigated, named as a long and narrow connection and short and wide connection. All specimens were fabricated from a grade G40.21-300W steel plate of 6.4mm (1/4 inch.) nominal thickness. As a result a reliability was conducted on a database consisting of 128 test results and 5 finite element analysis, four models, two consisting of the design equations currently used in North American Standards, an equation proposed by Hardash and Bjorhovde (1984), and an equation that shown by Driver (2004) to predict quite well the tension and shear block capacity of various connection types, were evaluated. Moreover, the equations in the AISC (1999) specification failed to predict the observed failure mode.

Topkaya (2004) [15] discussed Finite Element parametric studies on block shear failure of steel tension members and conducted test to develop simple block shear load capacity prediction equations that are based on finite element analysis. Over a thousand non-linear analysis were performed to identify the important parameters that influence block shear capacity. In addition the effects of eccentric loading were investigated. Based on the parametric study block shear load capacity prediction equations were developed. The predictions of the developed equations were compared with the experimental findings and were found to provide estimates with acceptable accuracy. The predicted equation was

$R_n = 0.48\ f_u\ A_{gv} + fu\ A_{nt}$.

Kara, Emre M.S., (2005) [16] a parametric study was conducted and over a thousand finite element analyses were performed to identify the parameters affecting the block shear failure in connections with multiple bolt lines and staggered holes. The quality of the specification equations were assessed by comparing the code predictions with finite element results. In addition, based on the analytical findings new equations were developed and were presented.

Padmapriya *et al.* (2010) [17] study 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 Universal Testing machine subjected to eccentric tensile load. From the test results, load Vs deflection behavior and the failure modes were studied. The actual load carried by the specimen was compared with the theoretical load carrying capacity predicted by International code provisions and with the load carrying capacity predicted by numerical investigation by ANSYS. An empirical equation was proposed to determine the load carrying capacity of the cold-formed steel angles and the predicted values agree with the experimental results.

Galasso (2011) [18] study investigated the block shear design equations as they had progressively changed from the 1978 provisions for Allowable Stress Design (ASD) to the 2005 provisions for Load and Resistance Factor Design (LRFD). Block shear strength capacities were calculated for multiple designs involving coped beams, angles, and structural tees. These analytical values were compared to physical test findings available in the literature. The results of this study compare the different strength predictions to one another, as well as benchmark the AISC provisions to actual physical testing strengths. The comparisons were also used to determine whether the ASD and LRFD specifications follow similar trends. Good agreement between the predicted capacities and the results from physical testing was observed for a majority of the geometries investigated. However, capacity predictions based on increasing the number of rows of bolts for a coped beam and changing the length of the outstanding leg for an angle or tee connection were found to disagree with the test results. A finite element study was also completed to further explore the influence of changing the length of the outstanding leg of tee connections because these geometries showed considerable disagreement between the calculated capacities and the test data.

Subuhan *et al.* (2012) [19] a parametric study was done over a number of finite element analyses performed by Subuhan and Jayabalan to identify the block shear failure of angle section connected in staggered pattern. The analysis was done using a 3D Finite element software ABAQUS. The quality of the block shear equations specified in IS code were modified by including the inclined line equations to it. The analyses done in ABAQUS for different angle sections connected in staggered manner were compared with the identical angle sections designed using modified design equations of IS code. Based on the results obtained the following conclusions were made- For positive stagger it was recommended that block shear path should follow the tension stagger line. For negative stagger it was recommended that block shear path should follow the stagger line represented in finite element analysis.

Clement *et al.* (2013) [20] have previously noted independent experimental evidence indicating the shear failure planes to lie midway between the gross and the net shear planes, termed the active shear planes. This paper presents the nonlinear contact finite brick element analysis results that confirm the location of the active shear planes, indicated by regions of maximum shear stresses. The finite-element analysis also found that shear stresses approach zero toward the free downstream end of the connection block. The veracity of the active shear area is further demonstrated in terms of the ability of the resulting block shear equation to predict the governing failure modes of test specimens consistently, in comparison with the equations assuming the gross and the net shear areas.

Rosentrauch *et al.* (2013) [21] the 2007 collapse of the I-35W Bridge in Minneapolis, Minnesota led to additional load rating requirements for gusset plates. The Federal Highway Administration provided the direct tension and block shear methods to assess tensile capacity for use in load rating, as well as a mechanics based method to assess shear capacity on critical planes. However, the three methods provide limited information about ultimate behavior for complex connections. Using finite elements, the authors modeled the connection from Whitmore’s 1952 study to evaluate the plate capacity and failure mechanism. The analysis showed that the direct tension method was conservative, but predicted neither capacity nor behavior. Model performance corresponded well with the critical block shear capacity load in the left diagonal, and shear capacity on the gross horizontal plane. Yielding on this plane began around critical bolt holes and spread to either side of the plate, forming the critical failure mechanism. From the literature review it is clear that none of the codes (American, European, Indian code) provide uniform equation for block shear failure in tension member with staggered bolted connections. AISC-LRFD and ASD, European code and Indian code predicts the inaccurate failure mechanism under block shear capacity. Researchers have suggested various block shear equations for multiple bolt lines and various staggered patterns that can be applied only to certain staggered configuration but lack uniformity. Literature review suggest that Professional factor (The ratio of the measured capacity, obtained either by laboratory testing or from a validated finite element analysis, to the capacity predicted by the equation using measured material properties and geometry.) provided in American standards and European code which represents the ability of a model to predict the block shear capacity either overestimates or underestimate the block shear failure.

IV. CONCLUSION

In this study block shear is potential mode of failure which has to be considered in designing the connection of steel structure. In bolts connection most of the connections are of diamond pattern because diamond pattern connection are much efficient to transfer the load to another member through it. As diamond formed by staggered bolting arrangement. Until now block shear strength equation or it failure is determined under laboratory test. There is no guideline that full fill the criteria of real life parameter on structure. In Indian Standard for steel structure IS: 800:2007, there is no guideline mentioned or provided for block shear strength and it failure. In 2007, I-35W Bridge in Minneapolis collapse due to additional load rated on gusset plates which leads mechanism of failure of that bridge. Since, in IS: 800:2007 block shear strength and its failure were not mentioned and one of the accident also occurred. So block shear strength dominated or should be consider in designing of steel connection because there is uncertainty between gross and net areas which should be determined as an equation in terms of yielding, rupture strength and any other required parameter terms.

V. REFERENCE

[1] G. L. Kulak and G. Y. Grondin, "Block Shear Failure in Steel Members — A Review of Design Practice," vol. 1, pp. 329–339.

[2] B. B. . Huns, G. Y. Grondin, and R. G. Driver, "Tension and shear block failure of bolted gusset plates," *Can. J. Civ. Eng.*, vol. 33, no. 4, pp. 395–408, 2006.

[3] H. I. Epstein and B. H. Thacker, "The effect of bolt stagger for block shear tension failures in angles," *Comput. Struct.*, vol. 39, no. 5, pp. 571–576, 1991.

[4] H. I. Epstein and F. S. Gulia, "Finite element studies of bolt stagger effects in tension members," *Comput. Struct.*, vol. 48, no. 6, pp. 1153–1156, 1993.

[5] T. J. Cunningham, J. G. Orbison, and R. D. Ziemian, "Assessment of American block shear load capacity predictions," *J. Constr. Steel Res.*, vol. 35, no. 3, pp. 323–338, 1995.

[6] L. H. Teh and M. E. Uz, "Block shear failure planes of bolted connections - Direct experimental verifications," *J. Constr. Steel Res.*, vol. 111, pp. 70–74, 2015.

[7] H. I. Epstein and R. Chamarajanagar, "Finite element studies for correlation with block shear tests," *Comput. Struct.*, vol. 61, no. 5, pp. 967–974, 1996.

[8] Y. Wu and G. L. Kulak, "Shear Lag in Bolted Single and Double Angle Tension Members - Report 187," no. 187, pp. 1–74, 1993.

[9] Z. Marković, D. Budevac, J. Dobrić, N. Fric, and M. Knežević, "Specific behaviour of thin-walled member joints with fasteners," *Gradjevinar*, vol. 64, no. 3, 2012.

[10] Orbison J.G., Wagner M.E., Fritz W.P., Tension Plane Behavior in Single-Row Bolted Connections Subjected to Block Shear, Journal of Constructional Steel Research 1999; 49; 225-39.

[11] Kulak G.L., Wu E.Y., Shear Lag in Bolted Angle Tension Members, ASCE Journal of Structural Engineering 2001; 123(9); 1144-52.

[12] Driver R.G., End Tear Out Failures of Bolted Tension Members, department of Civil and Environmental Engineering, University of Alberta, Edmonton, Alberta, Canada, 2004.

[13] M. Gupta and L. M. Gupta, "Limit state design of bolted steel angles under tension," *J. Struct. Eng.*, vol. 31, no. 4, pp. 265–274, 2005.

[14] Bino B.S. Huns, Gilbert Y. Grondin, and Robert G. Driver., NRC Research 2004; 17007-107 Avenue

[15] C. Topkaya, "Finite element modeling of block shear failure in coped steel beams," *J. Constr. Steel Res.*, vol. 63, no. 4, pp. 544–553, Apr. 2007.

[16] Kara Emre, "A NUMERICAL STUDY ON BLOCK SHEAR FAILURE OF STEEL TENSION MEMBERS," no. July, p. 113, 2005.

[17] R. Padma Priya, "Study on behaviour of bolted cold-formed steel angle tension members," *Indian J. Sci. Technol.*, vol. 8, no. 7, pp. 102–109, 2015.

[18] A. M. Galasso, "Parametric Exploration of Block Shear Formulations," 2011.

[19] A. M. N. Subuhan and P. Jayabalan, "Block Shear Analysis of Tension Member," vol. 1, no. June, pp. 200–203, 2012.

[20] D. D. A. Clements and L. H. Teh, "Active Shear Planes of Bolted Connections Failing in Block Shear," *J. Struct. Eng.*, vol. 139, no. 3, pp. 320–327, 2013.

[21] P. L. Rosenstrauch, M. Sanayei, and B. R. Brenner, "Capacity analysis of gusset plate connections using the Whitmore, block shear, global section shear, and finite element methods," *Eng. Struct.*, vol. 48, pp. 543–557, 2013.