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Block Shear Failure in a Steel Angle Tension Member – A Review of Design Practic

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***Abstract:*— Equations of block shear capacity vary significantly in various standards of steel structure design. A study has been conducted to examine the design equations according to standards from India, America, Europe, Canada, and Japan, as well as proposed improved equations for block shear capacity of tension member. To evaluate these standards and proposed improved equations, available test results for block shear failures in tension member are used. For tension member Indian standard, Eurocode, Japan Standard and Canadian standard are significantly conservative, however American Standard provides a good prediction of block shear capacity.**

***Index Terms* - Block shear failure; connections; steel angle; tension member**

I. INTRODUCTION

Angles are widely used as a tension member in steel structures such as trusses and lateral bracing systems. In most of the cases, single angles are connected by one leg only to the gusset plate. With the introduction of high strength bolts, lesser number of bolts required for the connection, which result in reduction in connection length. This reduction in connection length affects the failure mode of steel angle tension members. Experience and test results show that 'block shear' is potentially a failure mode for tension member, particularly when the connection is short [15].

In block shear failure, a block of the connected member is partially or fully driven from the remainder of the member. Although this mode of failure can occur in bolted/riveted and welded connections, it is more common in the former because of the holes, which reduced the cross-sectional area of the member. Typical examples of block shear shown in Figure 1.

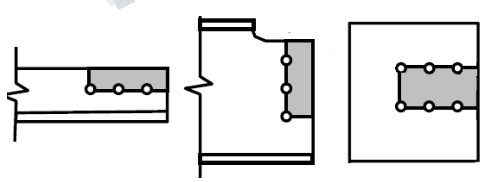

Figure 1 – Typical examples of Block shear

The provisions for block shear have been modified in almost every revision of various specifications. However, even the current provisions in various specifications are not the ideal reflection of experimental results, as predicted in the recent research in this area (shown in Table 2). An examination is done on various design standards provisions for the calculation of block shear capacity mentioned below.

II. CODES AND STANDARDS PROVISIONS

Design rules in various codes for block shear failure calculations based on a combination of yield shear resistance on the one plane and rupture strength on the other plane.
A review of available test results point out that the block shear failure modes seen in three important categories, gusset plate connections, tension members and the web of coped beams, are significantly different. This paper deals only with the tension member. Current specifications in each code relating to block shear are outlined and described below in Table 1. Table 2 shows the professional factor for the different standard. Professional factor is the ratio of the measured capacity, obtained either by laboratory testing or from a finite element analysis, to the capacity predicted by the equation using measured material properties and geometry [15].

Indian Code of Practice for General Construction in Steel; IS 800: 2007 [1] governed the block shear capacity based on yield strength of the gross shear area and rupture strength of the net tensile area and/or rupture strength of the net shear area and yield strength of the gross tensile area. The block shear capacity is taken as the least of the equation (1) or equation (2).

As shown in Table 2, equation (1) and equation (2) underestimate the block shear capacities of all the reported specimens and equation (2) governed the failure criteria. The average professional factor for all reported specimens is 1.36, and the associated standard deviation is 0.17.

Table 1 - Block shear design provisions

Specification or Standard	Block Shear Strength
Indian Standard (IS 800:2007): General Construction in Steel-Code of Practice	$P_u = \frac{A_{gv}F_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9A_{nt}F_u}{\gamma_{m1}}$ (1) $P_u = \frac{0.9A_{nv}F_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{gt}F_y}{\gamma_{m0}}$ (2)
American National Standard (ANSI/AISC 360-10): Specification for Structural Steel Buildings	$P_u = 0.60F_uA_{nv} + U_{bs}F_uA_{nt}$ (3) $P_u = 0.60F_yA_{gv} + U_{bs}F_uA_{nt}$ (4)
European Standard (EN 1993-1-8:2005(E)): Eurocode 3: Design of steel structures - Part 1-8: Design of joints	$P_u = \frac{F_uA_{nt}}{\gamma_{M2}} + \frac{F_yA_{nv}}{\sqrt{3}\,\gamma_{M0}}$ (5) $P_u = \frac{0.5\,F_uA_{nt}}{\gamma_{M2}} + \frac{F_yA_{nv}}{\sqrt{3}\,\gamma_{M0}}$ (6)
Canadian standard (CSA-S16-09): Limit States Design of Steel Structures	$P_u = \phi_u[U_tA_{nt}F_u + 0.6A_{gv}(F_y + F_u)/2]$ (7)
Architectural Institute of Japan (AIJ 1990): Standard for limit state design of structures	$P_u = F_uA_{nt} + \frac{F_yA_{nv}}{\sqrt{3}}$ (8) $P_u = F_yA_{nt} + \frac{F_uA_{nv}}{\sqrt{3}}$ (9)

where Pu is the block shear capacity; Agv is the gross area in shear; Anv is the net area in shear; Atg is the gross area in tension; Atn is the net area in tension; Fy is the yield strength of the material; Fu is the ultimate strength of the material; γm0 is the partial safety factor for failure in tension by yielding (γm0 = 1.10); γm1 is the partial safety factor against ultimate tension failure by rupture (γm1 = 1.25);Ubs is a reduction factor (Ubs = 1, tension stress is uniform ; Ubs = 0.5, tension stress is nonuniform); γM0 is the partial factor for resistance of cross-sections in tension (γM0 = 1.00); γM2 is the partial factor for resistance of cross-sections in tension to fracture (γM2 = 1.25); ϕu is a Resistance factor (ϕu = 0.75); Ut is the Efficiency factor depends on type of connection (Ut = 0.6 for angle section).

American National Standard ANSI/AISC 360-10 [2] has adopted a conservative model to predict the block shear strength, but it shows good predictions for the block shear capacity of the tension member. The AISC Specifications present equations to predict the block shear rupture strength depends on the assumption that gross yielding on the shear plane occur when tearing on the tensile plane commences if 0.6FuAnv exceeds 0.6FyAgv. The block shear capacity is taken as the minimum of equation (3) or equation (4).

The professional factor predicted by ANSI/AISC 360-10 range from 0.96 to 1.11 for equation (3) and 0.84 to 1.09 for equation (4). The corresponding average professional factor is 1.04 and 0.99 respectively. This indicates that ANSI/AISC 360-10 can provide good predictions for the block shear capacity of tension member.

The rules presented in Eurocode 3 [3] are based on the fundamental assumption that Block shear failure consists of failure in shear at the row of bolts along the shear face of the hole group accompanied by tensile rupture along the line of bolt holes on the tension face of the bolt group. Eurocode uses reduced yield shear resistance equal to F_y/√3. Block shear capacity for concentric loading given by equation (5) and for eccentric loading given by equation (6).

As shown in Table 2, Eurocode 3 provides the most conservative estimation for block shear in tension member. Equations (5) and (6), both underestimate the block shear capacities and Equation (6) governed the failure criteria for all the reported specimens. The average professional factor for all reported specimens is 1.81, and the associated standard deviation is 0.33.

Table 2 - Professional factor for different standard

Source	Type	No. of Tests	Professional factor								
			Eq. 1	Eq. 2	Eq. 3	Eq. 4	Eq. 5	Eq. 6	Eq. 7	Eq. 8	Eq. 9
Madugula et al. [6] 1988	Angles	12	1.02 (0.17)	1.24 (0.18)	0.96 (0.14)	0.84 (0.12)	1.32 (0.18)	1.66 (0.37)	1.11 (0.28)	1.20 (0.17)	1.11 (0.20)
Gross et al. [8], 1995	Angles	13	1.09 (0.35)	1.27 (0.22)	0.99 (0.16)	0.91 (0.26)	1.33 (0.33)	1.57 (0.55)	1.19 (0.39)	1.24 (0.27)	1.10 (0.56)
Orbison et al. [9], 1999	Angles	3	1.32 (0.13)	1.43 (0.16)	1.07 (0.09)	1.06 (0.09)	1.46 (0.11)	1.95 (0.24)	1.52 (0.21)	1.30 (0.08)	1.27 (0.13)
	Tees	9	1.30 (0.27)	1.50 (0.22)	1.11 (0.21)	1.06 (0.20)	1.50 (0.28)	1.93 (0.42)	1.48 (0.38)	1.35 (0.26)	1.28 (0.25)
Gupta et al. [14], 2003	Angles	2	1.32 (0.03)	1.39 (0.05)	1.06 (0.04)	1.09 (0.03)	1.56 (0.05)	1.94 (0.08)	1.38 (0.04)	1.42 (0.04)	1.24 (0.05)
Average Standard Deviation			1.21 (0.19)	1.36 (0.17)	1.04 (0.13)	0.99 (0.14)	1.44 (0.19)	1.81 (0.33)	1.34 (0.26)	1.30 (0.17)	1.20 (0.24)

Table 3- Professional factor for different proposed unified equation

Source	Type	No. of Tests	Professional factor								
			Eq. 10	Eq. 11	Eq. 12	Eq. 13	Eq. 14	Eq. 15	Eq. 16	Eq. 17	Eq. 18
Madugula et al. [6] 1988	Angles	12	0.75 (0.12)	0.76 (0.12)	0.78 (0.12)	0.88 (0.13)	0.87 (0.12)	0.87 (0.12)	0.87 (0.12)	0.84 (0.13)	1.01 (0.10)
Gross et al. [8], 1995	Angles	13	0.84 (0.16)	0.84 (0.19)	0.87 (0.18)	0.89 (0.23)	0.88 (0.26)	0.91 (0.11)	0.90 (0.12)	0.92 (0.22)	1.15 (0.51)
Orbison et al. [9], 1999	Angles	3	1.06 (0.09)	0.98 (0.09)	0.99 (0.09)	0.99 (0.09)	0.96 (0.07)	0.97 (0.09)	0.97) (0.09)	1.08 (0.10)	1.44 (0.09)
	Tees	9	1.06 (0.20)	0.98 (0.20)	1.00 (0.20)	1.01 (0.15)	0.98 (0.15)	1.00 (0.16)	1.00 (0.16)	1.08 (0.22)	1.34 (0.37)
Gupta et al. [14], 2003	Angles	2	0.92 (0.02)	0.94 (0.02)	0.94 (0.02)	1.06 (0.04)	1.07 (0.03)	0.96 (0.03)	0.97 (0.03)	1.05 (0.03)	1.49 (0.00)
Average Standard Deviation			0.90 (0.12)	0.90 (0.13)	0.91 (0.12)	0.97 (0.13)	0.95 (0.13)	0.94 (0.10)	0.94 (0.10)	1.00 (0.14)	1.28 (0.21)

Note: the number in parentheses is the standard deviation.

The Canadian design standard (CSA-S16-09) for steel structures [4] makes the assumption that the block shear capacity is governed by fracture of the net tension area combining with the failure of the gross shear area with the average yield and ultimate shear strengths. The block shear capacity governed by the equation (7).

The professional factor predicted by CSA-S16-09 range from 1.11 to 1.52. The corresponding average professional factor is 1.34. This indicates that CSA-S16-09 underestimate the block shear capacities of all the reported specimens.

The Architectural Institute of Japan (AIJ 1990) [5] provides two equations for predicting the block shear capacity. Equation (8) and (9) uses net areas on both the planes. As shown in Table 2, equation (8) and equation (9) underestimate the block shear capacities of all the reported specimens and equation (8) governed the failure criteria in most of the cases. The average professional factor for all reported specimens is 1.30, and the associated standard deviation is 0.17.

III. LITERATURE REVIEW

Over the past three decades, research has been carried out by eminent people to examine the block shear failure mode in tension member. Angles connected through one leg and tees connected with tension through their stems are considered to behave similarly and are therefore grouped together as a single section type [13]. Along with experimental studies, finite element methods as well as statistical studies have been employed in various research works. Experiments on riveted and bolted angle and tee connections have been studied and the main results are outlined below. Summaries of each research paper studied herein are described individually in the following section. Table 3 shows the professional factor for different proposed unified equation.

Madugula et al. [6] reported tension test on single angles were carried out in the United States for Bonneville Power Administration (BPA) in 1985, Western Area Power Administration (WAPA) in 1987, and by Nelson in 1953 for the British Constructional Steelwork Association. Out of the fifty-nine test results reported, for twelve specimens, the mode of failure was block shear and not tension failure on net section. These twelve tests were the earliest tests reported for block shear failure in angles. The authors summarized the provisions for computing the strength of bolted single angles tension member from five different specifications.
Epstein [7] reported the results of full-scale testing of double-row, staggered, and un-staggered bolt connections on pair of structural steel angles. Three specimens tested for each connection configuration and the average failure load was reported for 38-connection configuration. The number of variables was the size of angles, eccentricity of load, size of outstanding leg, bolt stagger, pitch, and shear length which leads to different modes of failure. The failure modes included block shear, predominantly block shear with some net section, predominantly net section with some block shear, net section rupture, and bolt shear plus block shear. Block shear the sole mode of failure observed in only 15 individual tests (three tests each of five series). Twelve test specimens had a staggered bolt arrangement and three test specimens had un-stagger arrangement with two bolt lines. Staggered bolt hole connection introduces another parameter in block shear failure.

Gross et al. [8] conducted full-scale tests on thirteen single-angle tension members with bolted end connections. Ten specimens were made of A588 Grade 50 steel and three specimens were made of A36 steel angles with two or more bolts in a single row at the end connection. The angles and their connections were designed such that a block shear failure mode would limit the load capacity. From the test, the author observed that the yield of shear plane occurred along the edge of the bolt holes opposite the block at the time of tension plane rupture. Experimental results were compared with those predicted by the American Institute of Steel Construction (AISC) Load and Resistance Factor Design (LRFD) and Allowable Stress Design (ASD) Specifications at the time. From the tests, the authors concluded that the AISC ASD block shear equation accurately predicted the failure loads. The AISC LRFD Specification were overestimated the nominal failure loads.

Orbison et al. [9] conducted tests on 12 tension member specimens that failed in block shear. Three specimens were single-angle tension member fabricated from A36 steel and nine specimens were WT sections. The effects of the variation of several parameters were presented. The number of connection variables was in-plane eccentricity, out-of-plane eccentricity, connection length, and bolt line edge distance perpendicular to the load line. The authors were observed in all the cases, failure occurred through rupture of the tension plane, accompanied by inelastic shear deformation along the gross shear plane. The angle specimens developed in and out-of-plane flexural deformations and WT specimens developed in-plane flexural deformations. After examining experimental results, the authors concluded: a) the ultimate block shear load capacities derived from the AISC LRFD and ASD block shear equations were in reasonable and conservative agreement with the experimental failure loads. b) The block shear load capacity of a connection is a function of the load developed by the gross shear plane area at the time rupture of the tension plane initiates at the bolt hole. Recommendations were given based on the ultimate load and the strain variation along the tension plane that was measured during the experiments.

Gupta et al. [10] conducted limited tests on single angle tension member on a relatively short connection with bolts placed along the standard gauge line. The authors pointed out that in most of the previous experimental data, the bolts were placed away from the standard gauge line and a good number of specimens violet the minimum specified limits on end distance, edge distance, pitch and minimum thickness of the sections. The authors concluded that the block shear failure mode can occur even when the end distance, edge distance, pitch within the minimum prescribed limits specified by the standards.

IV. IMPROVED DESIGN EQUATIONS

To account for a block shear failure phenomenon in a particular type of member, various proposals for modification to the equation had been proposed. The literature that are presented and proposed as unified models to predict block shear capacity of the tension member reviewed in the following discussion. Other models derived from specific types of members are therefore excluded from this paper.

Topkaya [11] conducted finite element simulations of bolted plates, angles, tees, and channel sections. The load applied either concentrically or with eccentricity (in-plane or out-of-plane) to assess the sensitivity of a variety of factors on block shear capacity. Von Mises yield criterion with isotropic hardening was used in true stress- true strain response. Newton-Raphson method was used to trace the entire nonlinear load-deflection response and failure load were assumed to be the maximum load reached during the loading history. Author concluded that, the block failure typically influenced by the Ultimate to yield ratio, connection length, and boundary conditions. In-plane and out-of-plane eccentricities were found generally to have only a small effect. Author proposed three equations based on the analysis performed to predict block shear load capacity:

The effective shear stress based on the ultimate-to-yield ratio and the connection length

$$P_u = \left(0.25 + 0.35\frac{F_u}{F_y} - \frac{Cl}{2800}\right) F_y A_{gv} + F_u A_{nt} \qquad (10)$$

The effective shear stress based on the ultimate-to-yield ratio

$$P_u = \left(0.20 + 0.35\frac{F_u}{F_y}\right) F_y A_{gv} + F_u A_{nt} \qquad (11)$$

For effective shear stress based on the ultimate strength

$$P_u = 0.48 F_u A_{gv} + F_u A_{nt} \qquad (12)$$

where Cl is the connection length (distance from the center of the leading bolt hole to the end of the plate).

Table 3 shows equation (10) gives un-conservative predictions with the average professional factor is 0.90 and the associated standard deviation is 0.12. The average professional factor for the equation (11) and (12) are 0.90 and 0.91, respectively, which shows these equations give un-conservative prediction.

Cunningham et al. [12] performed a statistical study on 77 individual specimens. In this study, test data were obtained from eight different sources. Geometric parameters, such as in-plane load eccentricity and the aspect ratio of the block had been investigated and proposed a series of equations that attempted to account for this geometric parameters. In all cases, an exponential series was considered, which consisted of the sum of a tensile force term and the shear force term. Using the experimental data and optimization procedure, the coefficients of the exponential series were determined. After some simplification, the resulting equation reported as:

$$P_u = 0.55 A_{nt} F_u + \left[1.55\left(\frac{l_{vn}}{l_{tn}}\right)^{-0.25} - 0.1 e_s\right] A_{nv} F_y \quad (13)$$

A review of experimental results revealed that most of the test exhibited, at failure, rupture along tension plane and plastic deformation (but not rupture) of the shear plane. Therefore, the authors incorporate a tensile force term using the ultimate tensile strength of the steel, and a shear force term using the tensile yield strength in exponential series. Using material strength combinations, the following equations were proposed:

Tension and shear yield:

$$P_u = 1.05 A_{nt} F_y + \left[1.45\left(\frac{l_{vn}}{l_{tn}}\right)^{-0.25} - 0.1 e_s\right] A_{nv} F_y \quad (14)$$

Tension ultimate and shear ultimate strength:

$$P_u = 0.7 A_{nt} F_u + \left[\left(\frac{l_{vn}}{l_{tn}}\right)^{-0.2} - 0.08 e_s\right] A_{vn} F_u \quad (15)$$

Tension yield and shear ultimate strength:

$$P_u = A_{nt} F_y + \left[\left(\frac{l_{vn}}{l_{tn}}\right)^{-0.2} - 0.08 e_s\right] A_{nv} F_u \quad (16)$$

where e_s is the in-plane shear eccentricity, l_{tn} is the net length of tension plane and l_{vn} is the net length of shear plane.

As shown in Table 3, the average professional factor in the equation (13), (14), (15) and (16) are 0.97, 0.95, 0.94 and 0.91, respectively, and the corresponding average standard deviation are 0.13, 0.13, 0.10 and 0.10, respectively, which shows these equations give the un-conservative prediction.

Driver et al. [13] conducted a reliability study on a database of 205 block shear tests from seventeen different research programs. For this study, as well as from the failure modes observed in the tests, the authors proposed a unified block shear equation as:

$$P_u = R_t A_{nt} F_u + R_v A_{gv} \left(\frac{F_y + F_u}{2\sqrt{3}}\right) \quad (17)$$

where R_t and R_v are tension area and shear area mean stress correction factors, respectively.

The proposed unified equation combines effective stresses on both the net tension area and the gross shear area. The professional factor predicted by Driver et al. is 1.00. This

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indicates that the equation (17) provides better predictions for the block shear capacity of angle and tee specimens.
Gupta et al. [14] reported an improved approach to adequate prediction of block shear capacity. In this study the effect of staggered as well as non-staggered bolt in single and double angles section were investigated. Nonlinear Finite element analysis was used to verify the available experimental results. Available experimental results, which follow all provisions regarding end distance, edge distance, and pitch, were considered for analysis. Author proposed improved equation based on the analysis performed to predict block shear load capacity:

$$P_u = F_u A_{nt} + \frac{F_y A^*_{gv}}{\sqrt{3}} \qquad (18)$$

where A^*_{gv} is the effective block gross shear area (Figure 2)

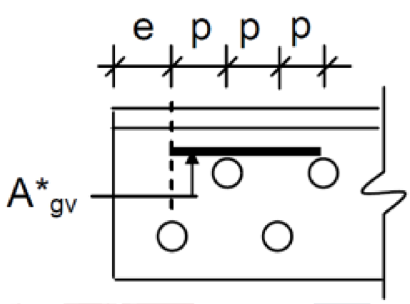

Figure 2 – Effective block gross shear area

Table 3 shows the average professional factor for the equation (18) is 1.28 and the associated standard deviation is 0.21. This indicates that the equation (18) provides conservative predictions for the block shear capacity of angle specimens.

V. SUMMARY, CONCLUSIONS AND RECOMMONDATION

This paper presents the state-of-art review of block shear failure in tension member. Equations available for the block shear capacity of the tension member of several design standards have been reviewed. This examination has recognized that the block shear capacity of steel tension member specified by Indian standard, Eurocode, Japan Standard and Canadian standard are significantly conservative, but American Standard provides a good prediction.

This review has identified the need for further studies of block shear capacity of the steel angle tension member. In particular, tension member with staggered and non-staggered bolt holes. In addition to this, it is observed that utmost results available for block shear failure in tension member are of bolted connections. Therefore, it is recommended that to carry out an experimental program to investigate the block shear capacity of the tension member with welded connection and proposed the unified equation for the same.

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