ISSN No: 2348-4845
International Journal & Magazine of Engineering, Technology, Management and Research
A Peer Reviewed Open Access International Journal

# Block Shear Failure in Tension Members

**R Ragul**
B. Tech Scholar
Department of Civil Engineering,
Siddhartha Institute of Engineering and Technology,
Vinobha Nagar, Ibrahimpatnam, Hyderabad,
Telangana 501506, India.

**Janardhana Reddy**
Assistant Professor
Department of Civil Engineering,
Siddhartha Institute of Engineering and Technology,
Vinobha Nagar, Ibrahimpatnam, Hyderabad,
Telangana 501506, India.

## *Abstract*

***Available test data on block shear behaviour of coped beams with double bolt-line connections are quite limited, and earlier investigations found that the test block shear capacities could not be accurately predicted by existing design rules which deal with this failure mode in an inconsistent manner. To address this, a comprehensive investigation focusing on the block shear behaviour of coped beams with double bolt-line connections was reported in this paper. The research commenced with 17 full-scale tests considering the test parameters of web block aspect ratio, out-of-plane eccentricity, connection rotational stiffness, and bolt stagger. Two specimens were found to fail by local web buckling, and the remaining 15 specimens failed by block shear. Three typical block shear failure modes were observed at ultimate load, namely, whole block tear-out (WBT), tensile fracture (TF), and tensile fracture followed by whole block tear-out (TF–WBT). The influences of the considered test parameters on the failure mode and block shear capacity of the test specimens were thoroughly discussed. The test results were then compared with existing design rules to evaluate the consistency and accuracy of the major standards, and it was found that these standards led to inconsistent test-to-predicted ratios and tended to be conservative. Summarising all available test data, including the current tests and those previously conducted by other researchers, a reliability analysis was conducted to further examine the level of safety of the major standards. Design recommendations were finally proposed aiming to achieve reliable yet economical design approaches with consistent safety levels.***

## Introduction

In steel construction, beams are often coped (cut away) atthe flanges to provide clearance for the framing beams or to maintain the same elevation for intersecting beams. The cope can be at the top (Fig. 1), the bottom, or both flanges near the connection in order to facilitate construction. The ends of the coped beam are commonly connected to the web of the girder by double clip angles.

The clip angles may be either bolted or welded to the web of the coped beam. One of the potential modes of the failure of the coped beam with a clip angle connection is the block shear of the beam web material. Block shear is a phenomenon of rupturing or tearing, where a block of material is torn out by a combination of tensile and shear failure [1]. Fig. 2 shows the potential block shear failure

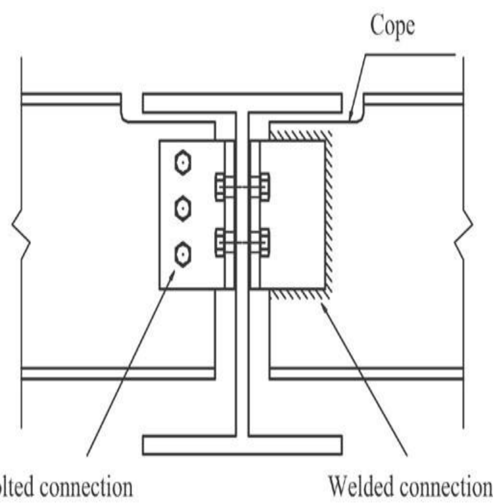



**Fig. 1. Schematic of a bolted or welded connection on a coped beam.**

Mode in a coped beam at the shear connection. The web block

ISSN No: 2348-4845
International Journal & Magazine of Engineering, Technology, Management and Research
A Peer Reviewed Open Access International Journal

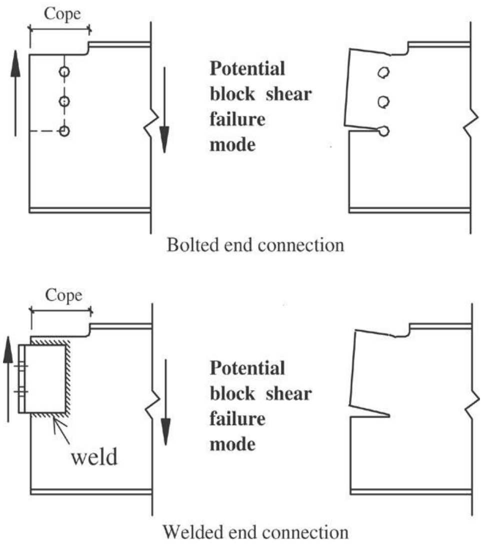



**Fig. 2. Potential block shear failure in a coped beam.**

*Note*: The block shear failure generally consists of shear yielding on the gross area of the shear face and tensile rupture along the net area of the tension face.Connections depending on the details of the connection. The block shear failure mode appears in different types of bolted structural members such as gusset plates, coped beams, angles, or tee-sections ([2–3] etc.).

The block shear failure mode of coped beams was first identified by Birkemoe and Gilmor [1]. In their tests, the coped beam failed by the tearing of the web as a block of material at the shear connection. The authors suggested that the failure model of block shear was provided by a combination of tensile and shear stresses acting over their respective areas (across plane AA and plane BB, respectively) as shown in Fig. 3. Yura et al. [4-6] conducted a series of twelve coped and uncoped beam tests with bolted double clip angle connections. Among eight coped beam tests, three exhibited failure of the block shear type. The specimens with two lines of bolts had a lower capacity than desired. A further investigation by Ricles and Yura [7-8] was carried out to examine the block shear failure mode of coped beams for bolted connections. The test parameters included the end and edge distance, bolt arrangement, and the type of holes. All seven coped beam specimens failed in the block shear mode and the web buckled at the cope in four of them. Based on the test and the finite element analysis results, the authors suggested assuming shear yielding for the vertical side of the web and tensile fracture for the horizontal side (perpendicular to the applied reaction force at the coped end) in evaluating the block shear strength of the connection [9-10]. It was further noted that the shear yielding that occurred along the gross vertical area of the web was based on the experimental observation. This implied that the connection capacity was the sum of a triangular normal stress on the net area of the tension face and shear yielding on the gross shear area as shown in Fig. 4. The proposed capacity equation is as follows:

$$P = 0.5F_u A_{nt} + 0.6F_y A_g\text{v} \quad (1)$$

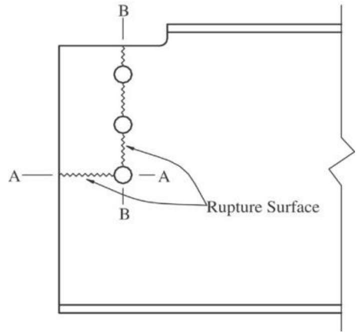



**Fig. 3. Block shear model of failure identified by Birkemoe and Gilmor [1].**

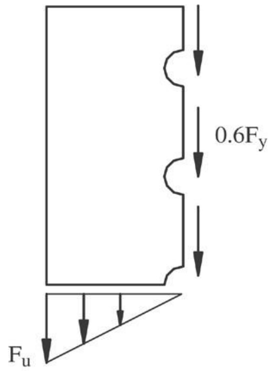



**Fig. 4. Block shear model proposed by Ricles and Yura [9]. where:**

| | |
|---|---|
| *P* | is the ultimate connection capacity; |
| $F_u$ | is the tensile strength; |
| $F_y$ | is the yield strength; |
| *Ant* | is the net tension area, and |
| *Agv* | is the gross shear area. |

ISSN No: 2348-4845
International Journal & Magazine of Engineering, Technology, Management and Research
A Peer Reviewed Open Access International Journal

Aalberg and Larsen [10] examined the behavior of coped I-beams with bolted end connections fabricated from highstrength steel and normal structural steel. Identical failure modes were observed for both normal and high-strength steels except that the connection ductility was reduced for the highstrength steel specimens. A comprehensive review of block shear issues was conducted by Kulak and Grondin [11,12]. They revisited the block shear failure mode in different cases of coped beams, gusset plates, and angles. The review showed that the failure modes were significantly different in two important categories, namely: gusset plates and the web of coped beams. Taylor [13] discussed the issue of the lack of experimental data for the block shear strength of coped beams with welded connections. Most recently, Franchuk et al. [14] conducted an extensive experimental program consisting of seventeen tests to investigate the block shear behavior of coped beams with bolted connections. The test results indicated that magnitudes ofTension and shear areas significantly affect connection capacity.

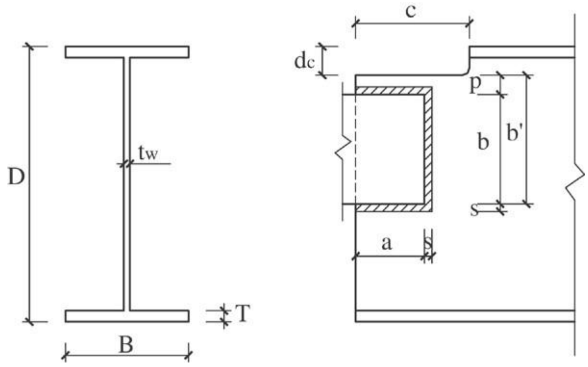



Fig. 5. Schematics of the test specimen.

The test results also substantiated the previous experimental observation that shear yielding on the gross (vertical) area should be used in the block shear design of coped beams. The latest research was performed by Topkaya [15]. A finite element parametric study on the block shear failure of steel tension members was carried out to develop a block shear capacity prediction equation. The parametric study was conducted to identify the important parameters, such as ultimate-to-yield ratio, connection length, and boundary conditions.

Based on the above discussion, it can be seen that all of the available experimental and analytical studies on the block shear of coped beams concentrated on bolted end connections. Therefore, the main objective of this study is to provide experimental data for the block shear strength and behavior of coped beams with welded clip angle connections. The evaluation of the ultimate strength of the test specimens using current design codes will also be presented.

## II. Experimental program

### Test specimens

The purpose of the experimental program was to examine the block shear failure strength and behavior of a coped beam web with a welded clip angle connection. A total of ten fullscale tests were conducted in the experimental program. The test parameters included connection geometry and cope details such as the aspect ratio of the clip angles, the tension and shear area of the web block, web thickness, beam section depth, cope length, and connection position. The test was conducted individually at each end of the 3.3 m long test beam. The schematics and details of the specimens are shown in Figs. 5–7. These five test beams were fabricated from three different section sizes, including universal beam UB406 × 140 × 46, UB457 × 191 × 74, and UB356 × 171 × 67 [22]. Grade 43 steel conforming to BS 4360 [23] was used for the beams. All of the double clip angles were fabricated by 16 mm steel plates conforming to BS 4360 [23] Grade 50 to provide the required connection dimensions. The angles were designed to provide enough strength for the connections and, at the same time, to minimize the in-plane rotational stiffness in order to simulate a simply supported boundary condition. The double angles were welded to the web of the beams.

The nominal and measured dimensions of the beam sections and the connection details are shown in Tables 1 and 2, respectively. These tables should be read in conjunction with Fig. 5. The test beams were each designated by a letter (A through E), and each end connection was designated by a number, 1 or 2,

ISSN No: 2348-4845
International Journal & Magazine of Engineering, Technology, Management and Research
A Peer Reviewed Open Access International Journal

respectively. Each specimen was also designated according to the beam type, and the testing variable was assigned to facilitate the identification. For example, A1-406r3 represents beam A, connection 1, and UB406×140× 46 section with an aspect ratio of around 3. Fig. 6 illustrates the

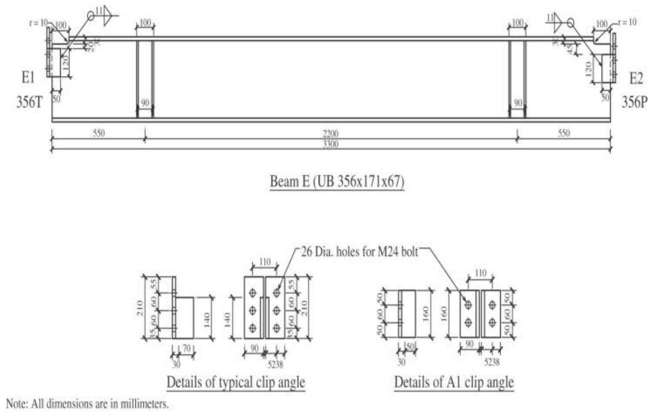
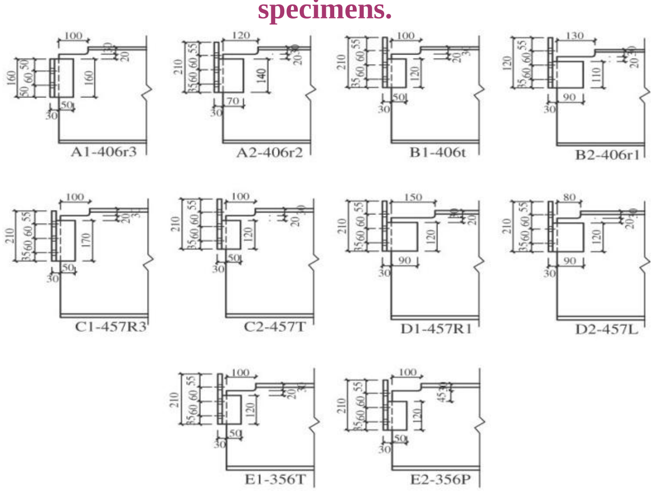



**Fig. 6. Overall design dimensions of the test specimens.**




Note: All dimensions are in millimeters.

**Fig. 7. Connection details of all of the test specimens.**

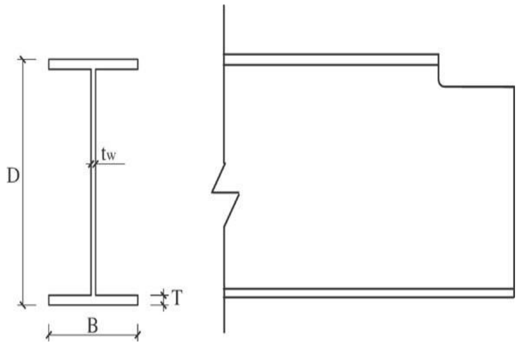



**Fig.8. typical overall design dimensions of the coped beam specimens and the clip angles, and Fig. 7 presents the connection details of all of the specimens.**

As summarized in Table 3, the test parameters included the aspect ratio of the clip angles, the tension and shear area of the web block, web thickness, beam section depth, cope length, and connection position. Only one parameter was varied in each group of tests. The specimens with various configurations were carefully designed to fail in the block shear mode by the current design practice. Note that for the aspect ratio series the design capacities of the related tests were nearly identical for the purpose of comparison. The top flange was coped for all specimens. In general, the cope dimensions were fixed, with the cope depth extending 30 mm below the top flange and the cope length extending 50 mm away from the end of the clip angles except for the specimens that were employed to study the effect of cope length.

In order to obtain the material properties, tension coupon tests were carried out. Tension coupons were prepared from the web and the flange of the to the ASTM A370 standard [24]. An extensometer with a 50 mm gauge length was used to measure the strain in the coupon and the load was obtained as a read-out of the testing machine. Pairs of strain gauges were mounted on both faces of the coupons in order to determine the modulus of elasticity ($E$) of the steel in the elastic range. The static readings were taken on the yield plateau, along the strain hardening curve, and near the ultimate tensile strength.

**Table 1**
**Cross-sectional dimensions of the test beams**

| Beam designation | Beam serial | $t_w$(mm) | $T$ (mm) | $D$ (mm) | $B$ (mm) | Note |
|---|---|---|---|---|---|---|
| A, B (Beam406) | UB406 × 140 × 46 | 6.8 | 11.2 | 403.2 | 142.4 | Nominal |
| | | 6.8 | 11.1 | 404.2 | 140.9 | Measured |
| C, D (Beam457) | UB457 × 191 × 74 | 9.1 | 14.5 | 457.0 | 190.4 | Nominal |
| | | 9.2 | 14.2 | 456.3 | 189.1 | Measured |
| E (Beam356) | UB356 × 171 × 67 | 9.1 | 15.7 | 363.4 | 173.2 | Nominal |
| | | 9.1 | 15.2 | 362.6 | 171.5 | Measured |

## III. Test results

### Material test results

The tension coupon test results for all specimens are summarized in Table 4, and the results for clip angles are included as well. Since the block shear failure was mostly associated with the beam web, the mean values of the coupons from the web of test beams were used as the basic material properties for the specimens. The average static yield strength for the specimens ranged from 304.1 to 371.6 MPa, whereas the average static ultimate strength ranged from 442.2 to 487.7 MPa. Although Grade 50 steel was assumed in the design of the clip angles, it was believed that Grade 43 steel was used instead for the clip angles based on the tension coupon test results as illustrated in Table 4. The material properties for S275 and S335 (EN10025 -2:2004) are also included in the table for comparison purposes.

### General

The test results are summarized in Table 5, where the static values of the ultimate load are reported, and the connection reactions and moments were calculated from static equilibrium. For simplicity, only the letter and the first number in the specimen designation were used to identify the specimens (for example, specimen A1 instead of specimen A1-406r3). The moment developed in the connection was found to be small compared with the yield moment of the beam. In general, there were two kinds of failure modes in the tests: the block shear of the beam web with tension fractures underneath the clip angles and local web buckling near the end of the cope. Two specimens failed in the block shear mode while six specimens ultimately failed in local web buckling near the cope due to combined shear and bending. Although the final failure mode of the six specimens was web local buckling, it was observed during the tests that these specimens exhibited a significant block shear type deformation prior to reaching their final failure mode. In the block shear cases, necking of the tension area in the web underneath the clip angles was observed before the web fractured abruptly. The reduction in web thickness in the tension region is also included in Table 5. A tensile crack developed in the web along the bottom of the welded clip angles rather than a complete web block tear-out. No signs of shear fracture along the vertical plane were observed. As to the remaining two specimens, the test loads exceeded the capacity of the testing frame; hence, these two tests were terminated at the safe load of the test set-up.

Yield lines were usually initiated in the web either underneath the welded clip angles at the extreme beam end or near the cope end. Eventually, the yield lines extended through the web block and could be obviously observed below the welded clip angle and across the cope end. These yield lines indicated that significant deformation had occurred in these regions. Yield lines could also be found in the shear area near the welded clip angles in most cases.

The results showed that the web plate buckled to various extents at the cope end even though the flange near the cope was braced against lateral movement. Severe compression and shear yielding was observed near the cope end before the specimens reached the ultimate loads. This indicated that local web buckling was also a potential failure mode for coped beams with welded clip angle connections. This failure mode was also observed by Ricles and Yura [9] in their coped beam tests. They pointed out that the high horizontal compressive stresses in the web could cause the web to buckle at the cope.

### Load deflection behavior

Typical applied load versus deflection curves are shown in Fig. 13. Plotted on the horizontal axis is the net deflection at the load point, which was determined by taking the difference between the displacement under the applied load and the displacement of the clip angles; hence, this net deflection excluded the displacements due to bolt bearing and slippage. Thus, the overall behavior of the test beams can be presented. Generally, similar behavior was observed of the specimens with regard to the two kinds of failure modes. As the load was applied, yielding first appeared in the web underneath the welded clip angles or near the cope end. Nonlinear behaviour then commenced. When the load continued to increase, shear yielding developed gradually along the vertical area near the welded angles. High flexural stresses in the upper region of the web increased and excessive deformation developed. The web block deformed significantly before the web plate near the cope end became distorted or buckled inelastically. Subsequently, for specimens C2 and E1, tension fractures developed abruptly underneath the clip angles, and the load therefore decreased significantly. Continued loading caused further opening of the cracks. As to specimens A1, A2, B1, B2, D1, and E2, at the ultimate load level severe compression as well as shearing caused local web buckling at the cope end, and the load subsequently descended.

## IV. Discussion of the test results

### Failure mode

Previous studies focused on the block shear failure of a coped beam web with bolted connections. For bolted end connections, a reduction in the cross-sectional area due to the presence of holes has adverse effects on the strength of the connections. The concentration of stress at the bolt holes initiated fracturing, and contributed to the block shear failure mode. For welded end connections, since there is no reduction in gross section area due to bolt holes, the behavior and failure mode of the connection would be different.

As described before, two kinds of failure modes were observed in the tests, namely; block shear failure and local web buckling at the cope end. Block shear failure was exhibited in a partial tear-out of the web block with a tension fracture underneath the clip angles. The tension fracture was triggered at the extreme end of the beam web, and necking of the web plate was observed before a sudden rupture. Shear yielding rather than shear fracture was observed along the vertical plane of the shear area. Shear ultimate strength was difficult to achieve since the shear deformation was relatively small compared with the tension deformation. Hence, tension fracture was reached prior to shear fracture due to the significant deformation in the area of tension. Another potential failure mode was local web buckling near the end of the cope. High compressive stresses and shear stresses were localized in the web near the cope because of the combination of bending and shear in the reduced section; hence resulting in extensive yielding and consequently inducing local web buckling in that region.

As mentioned before, lateral bracing at the top flange near the cope was provided to improve the lateral stability of the test beams. However, various degrees of web buckling were still observed in most cases, and six specimens among them ultimately failed in the local web buckling mode. In fact, in previous studies, such as those by Birkemoe and Gilmor [1] and Ricles and Yura [9], local web buckling was also observed in cope beam tests for block shear with bolted connections. For those specimens that failed due to local web buckling, excessive deformation developed around the web block at a high load due to significant yielding in the tension and shear areas. However, the deformation was insufficient to induce fracturing even though the ultimate tensile stress was attained in the area of tension. On the other hand, severe compression and shear yielding accumulated near the cope end, thus eventually resulting in local web buckling. Nevertheless, observations showed that these specimens exhibited a deformation of the block shear type prior to reaching their ultimate loads with local web buckling failure.

### Effect of aspect ratio

Aspect ratio has been defined as the ratio of vertical shear area ($b^0$) to the horizontal tension area ($a$) of the web block, as indicated in Fig. 5. Note that the design capacities of the related specimens are nearly identical for the purpose of comparison. The aspect ratio was examined in two series of tests. In the first series, specimens A1, A2, and B2 had an aspect ratio of 3.6, 2.3, and 1.4, respectively. Their final failure mode was local web buckling, as shown in Table 5. For specimens A2 and B2, the aspect ratio varied from 2.3 to 1.4 while the capacity of A2 was 12% higher than that of B2. Although specimen A1 had the largest aspect ratio, it was worth noting that the double clip angles used in this specimen were different from those of the other specimens due to a fabrication error. This had an adverse effect on the capacity of the specimen. Since the bolted leg of the clip angle was fabricated shorter for the arrangement of the bolts, the boundary condition of the pin support that was simulated by the clip angle was influenced to some extent. This meant that the position of the pin support for the beam specimen was therefore lowered, while the rotational stiffness of the joint was reduced. Since the negative moment that developed in the connection at the beam end was due to a slight fixity in connection, the negative moment of A1 in the connection was smaller than that of A2 and B2, as shown in Table 5. Hence, it was believed that the reduction in the capacity of specimen A1, which failed because of the local web buckling, was attributable to the influence of the clip angle. The results of the finite element analyses that will be discussed in the companion paper [25] also indicate that specimen A1 would have a larger capacity than specimen A2 by using double clip angles similar to those of the others. For specimens C1 and D1 in the second series, the aspect ratio was 3.8 and 1.6, respectively. Although the ultimate failure of C1 did not occur before the test was terminated at the safe load of the test set-up, it was believed that C1 would have a much larger capacity than D1 from the trend of the load deflection curve, which can be observed in Fig. 17.

In addition, further observations of the yield lines also led to an interpretation of the effect of the aspect ratio on the deformation of the web. For specimen D1, many yield lines were observed in the beam web near the bottom of the clip angle as shown in Fig. 11(d). This might indicate that the web material in this region had been subjected to excessive deformation caused by local bending due to the connection rotation. On the other hand, for specimen C1, there were significantly fewer yield lines, as shown in Fig. 11(b). Nevertheless, it can be seen that as the aspect ratio decreased, local bending of the web near the clip angle due to the connection rotation was more significant. In fact, the local bending would cause high compressive stress to the web material near the end of the cope corner and hence might lead to a failure in the local buckling of the web.

## V. Conclusions

Although many experiments have been conducted to study tension and shear block failure in gusset plates, our understanding of the progression of fracture on the tension and shear faces is still lacking. As a result, there exists confusion about the failure progression, and failure models such as the one presented in the AISC (1999) design specification predict failure modes that are inconsistent with experimental observations. To some extent, the models used in the current design standards fail to capture the observed failure mode for tension and shear block failure.

A finite element model was developed to study the progression of tension and shear fracture in bolted gusset plates. A simple analysis technique, consisting of removing elements as fracture progresses, was developed to model ductile rupture. Tension and shear rupture criteria were proposed to model the tension and shear failures. The finite element model was validated by comparing analysis results with results from gusset plate tests. The validated finite element procedure was then used to expand the database of test results to include a larger number of long connections, larger pitch distance, and larger gauge distance.

All the test results reported in the open literature and the finite element analyses conducted in this investigation indicate that tension fracture always occurs prior to shear rupture. Furthermore, except for unusually long and narrow connections, the full capacity of the connection is reached by the time tension rupture takes place. This supports capacity prediction models that incorporate rupture on the net tension area plus a contribution from the shear area of the concentric connection.

A reliability analysis was conducted on a database consisting of 128 test results and 5 finite element analysis results. Four models, two consisting of the design equations currently used in North American

ISSN No: 2348-4845
International Journal & Magazine of Engineering, Technology, Management and Research
A Peer Reviewed Open Access International Journal

standards, an equation proposed by Hardash and Bjorhovde (1984), and an equation that was shown by Driver et al. (2004) to predict quite well the tension and shear block capacity of various connection types, were evaluated. A comparison of predicted capacities and test results indicated that the latter two equations provided a generally good prediction of the test results, whereas the equations in the two design standards underpredicted the capacities by a significant margin. Moreover, the equations in the AISC (1999) specification failed to predict the observed failure mode. All four models provided low COVs, varying from 0.063 to 0.074. Because of the high level of conservatism in the current design equations, a resistance factor of 0.90 results in a safety index of 4.5. The proposed equation, as a consequence of the mean professional factor of 0.98, requires a resistance factor of 0.74 to provide the same level of safety.

Because of its simplicity, its ability to accurately predict the tension and shear block capacity of gusset plates, its reflection of experimental observations of the failure mode, and its consistency with a previously proposed unified equation suitable for various connection types, the model expressed by eq. [12] is recommended for design. A resistance factor of 0.75 in eq. [12] provides a safety index of 4.37.

## References

[1] Birkemoe PC, Gilmor MI. Behavior of bearing critical double-angle beam connections. Engineering Journal, AISC 1978;15(4):109–15.

[2] Hardash S, Bjorhovde R. New design criteria for gusset plates in tension. Engineering journal, AISC 1985;22(2). Second quarter.

[3] Epstein HI. An experimental study of block shear failure of angles in tension. Engineering journal, AISC 1992;29(2). Second quarter.

[4] Epstein HI, Stamberg H. Block shear and net section capacities of structural tees in tension: Test results and code implications. Engineering Journal 2002;39(4):228–39. Fourth quarter.

[5] Epstein HI, McGinnis MJ. Finite element modeling of block shear in structural tees. Computers and Structures 2000;77(5):571–82.

[6] Franchuk CR, Driver RG, Grondin GY. Experimental investigation of block shear failure in coped steel beams. Canadian Journal of Civil Engineering 2003;30(5):871–81.

[7] Orbison JG, Wagner ME, Fritz WP. Tension plane behavior in single-row bolted connections subject to block shear. Journal of Constructional Steel Research 1999;49:225–39.

[8] Yura JA, Birkemoe PC, Ricles JM. Beam web shear connections: An experimental study. Journal of the Structural Division, ASCE 1982; 108(ST2):311–25.

[9] Ricles JM, Yura JA. Strength of double-row bolted-web connections. Journal of Structural Engineering, ASCE 1983;109(12):126–42.

[10] Aalberg A, Larsen PK. Strength and ductility of bolted connections in normal and high strength steels. In: Proceedings of the seventh international symposium on structural failure and plasticity. 2000.

[11] Kulak GL, Grondin GY. Block shear failure in steel members — a review of design practice. In: Leon RT, Easterling WS, editors. Proceedings of fourth international workshop on connections in steel structures IV: Steel connections in the new millennium. Chicago (Ill): AISC; 2000. p. 329–39.

[12] Kulak GL, Grondin GY. AISC LRFD rules for block shear in bolted connections — a review. Engineering Journal, AISC 2001;38(4):199–203. See also “Errata and addendum”, Engineering Journal, AISC 39(2).

[13] Taylor JC. More work is required. In: Leon RT, Easterling WS, editors. Proceedings of fourth international workshop on connections in steel structures IV: Steel connections in the new millennium. Chicago (Ill): AISC; 2000. p. 33–40.

[14] Franchuk CR, Driver RG, Grondin GY. Block shear failure of coped steel beams. In: Proceedings of 4th structural specialty conference of the Canadian society for civil engineering. 2002.

[15] Topkaya C. A finite element parametric study on block shear failure of steel tension members. Journal of Constructional Steel Research 2004; 60(11):1615–35.
[16] American Institute of Steel Construction (AISC). Load and resistance factor design specification for structural steel buildings. Chicago (IL, USA); 1999.
[17] Canadian Standards Association (CSA). CAN/CSA-S16-01 Limit states design of steel structures. CSA; Toronto (ON, Canada); 2001.