The 2017 World Congress on
Advances in Structural Engineering and Mechanics (ASEM17)
28 August - 1 September, 2017, Ilsan(Seoul), Korea

Numerical studies on block shear failure of bolted high strength steel angles

Michael C H Yam[1),4)], *Binhui Jiang[2)] and K F Chung[3),4)]

[1), 2)] *Department of Building and Real Estate, The Hong Kong Polytechnic University, Hong Kong, China*
[3)] *Department of Civil and Environmental Engineering, The Hong Kong Polytechnic University, Hong Kong, China*
[4)] *Chinese National Engineering Research Centre for Steel Construction (Hong Kong Branch), The Hong Kong Polytechnic University, Hong Kong, China*
[2)] *b.jiang@polyu.edu.hk*

ABSTRACT

This paper presents a numerical study on the block shear behaviour and failure of bolted High Strength Steel (HSS) angles, and the effects of steel grade (Q690/Q960) were examined through parametric analysis. The performance of normal steel (S275) angles was also analysed for comparison purpose. The numerical model was calibrated through block shear tests in literature. The results show that all the specimens failed in block shear with a combination of rupture at the tension plane and yielding of the shear plane. It was found that although the angles of steel Q690 and Q960 have lower ductility than that of steel S275, they have sufficient ductility to enable the shear stress in the shear plane to reach 60% of the tensile strength of the material at the time of failure. Finally, comparisons between predictions of the American standard AISC-2010, Eurocode 3, Canadian standard CSA S16-09 on block shear strength and the analysis results were conducted. The results indicate that AISC-2010 and Eurocode 3 give conservative predictions of the block shear strength of both bolted normal steel and HSS angles. On the other hand, the Canadian standard CSA S16-09 overestimates the block shear strength of bolted HSS angles, while its predictions of the block shear strength of bolted normal steel angles may be either conservative or unsafe.

1. INTRODUCTION

The use of High Strength Steel (HSS) in civil engineering structures has been receiving more attention recently. In particular, tension members, such as angle sections, fabricated using HSS can significantly reduce the size of the member sections

---

1) Professor
2) Research Fellow
3) Professor

and provide significant savings in delivery cost and material consumption during steel production. However, the low ductility and high yield-tensile ratio of HSS reduce the material's ability to redistribute stresses from higher stressed areas, such as the vicinity of bolt holes, to lower stressed areas. Hence, the block shear behaviour of HSS angles may be different from that of normal steel angles and researches on block shear of bolted HSS angles are needed.

Although extensive studies have been carried out on block shear effects on bolted angles of normal steel (Chesson 1959, Epstein 1992, Kulak 2001, Topkaya 2004 and others), researches on block shear behaviour of bolted HSS angle especially with steel grade of Q690 or higher are still limited. Kouhi (1992) experimentally studied the tensile strength of bolted angles of HSS with yield strength of 640 MPa. Gross (1995) conducted experimental studies on the block shear failure of bolted angles of steel A588 grade 50 (the yield strength is around 430 MPa), and found that block shear failures in all test specimens involve rupture of the tension plane and yielding of the shear plane. Kim (1999) studied the effects of the ratio of ultimate strength to yield strength on the bearing capacity of bolted connections. Ling (2007) investigated the block shear behaviour of gusset-plate welded connections in tubes of very high strength, where the nominal yield strength was 1350 MPa. It was concluded that the block shear resistance of the test specimens was overestimated by the main codes. Clements and Teh (2012, 2013) conducted experimental and numerical studies on block shear performance of cold formed steel angles and proposed an equation to calculate block shear capacity of the angles.

In this paper, a numerical model was developed to study the block shear behaviour and failure of bolted HSS angles with steel grade of Q690 and Q960. For comparison, angles of grade S275 steel were also analysed in the study. The developed numerical model was validated through test results found in literature. Finally, code design equations of block shear capacity were evaluated.

2. SPECIMEN DIMENSIONS

Fig. 1 shows the dimensions of the proposed specimens. The angle is connected to a gusset plate at each end through a line of bolts. The angle section is 85*65*6 with the short leg being connected. The bolt diameter is 22 mm, and diameter of the bolt holes is 24 mm. The thickness of the gusset plate is 16 mm and its steel grade is the same as the connected angle. Tensile load is applied to the end of each gusset plate.

A total of 6 specimens were designed with variations in parameters of steel grade and number of bolts (Bt). Details of the specimens are listed in Table 1. The first letter of the specimen label A, B or C represents that the steel grade of the angle is S275, Q690 and Q960 respectively.

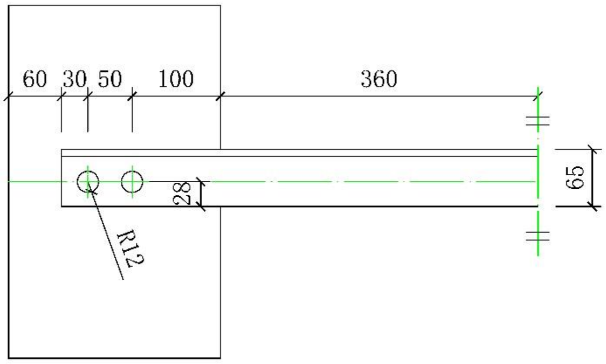


Fig. 1 Dimensions of specimen (mm)

Table 1 Details of specimens

Specimen	Steel	Bolt no.
A-Bt2	S275	2
A-Bt3	S275	3
B-Bt2	Q690	2
B-Bt3	Q690	3
C-Bt2	Q960	2
C-Bt3	Q960	3

3. NUMERICAL MODEL

The numerical model was built using the commercial finite element (FE) software ABAQUS (2014). Solid elements with reduced integration (C3D8R) were used to model the angle, bolt and gusset plate, as shown in Fig. 2. General contact was defined for the interactions between the surfaces of contact. The normal contact behaviour was considered to be “hard” contact and penalty friction formulation with a friction coefficient of 0.2 was adopted for the tangential contact behaviour. The surfaces of the bolts and the bolt holes were in contact before loading. Displacement was applied to the left end of the gusset plate and the right end of the angle was considered as symmetry plane. Since the main purpose of this numerical model is to analyse the failure mode and the ultimate strength of the angle specimens, while the behaviour in the load declining stage after failure initiation is not a main focus, general static analysis method was adopted to save computing cost.

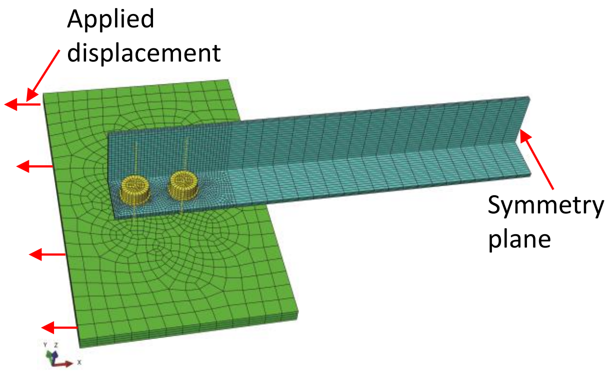


Fig. 2 Finite element model

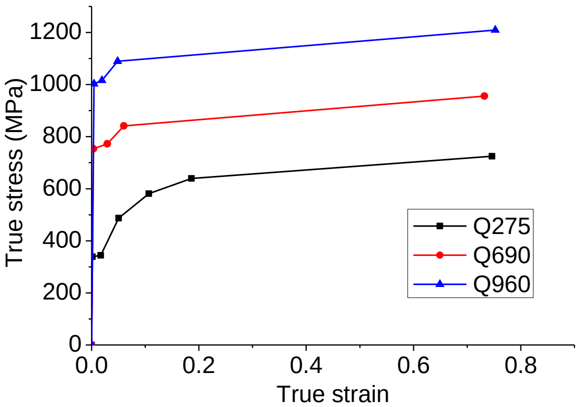
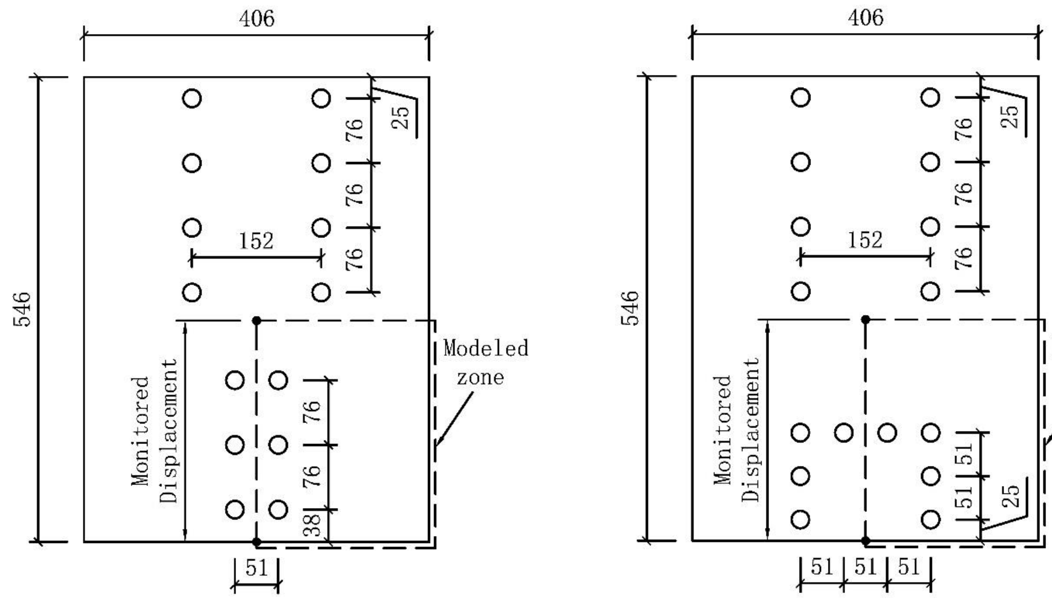


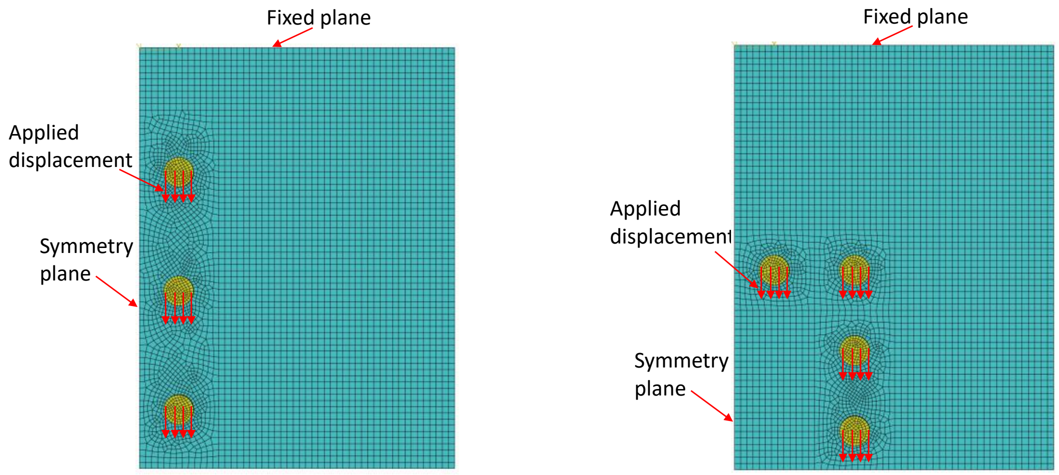
Fig. 3 True stress-strain curves of steel materials

An isotropic elastic-plastic model with von Mises yield criterion was adopted for the steel material. The stress-strain curves of the steel materials which were obtained from coupon tests are shown in Fig 3. The elastic modulus of steel was taken as 220,000 MPa for the three steel grades. To model the failure of material, ductile damage model with element removal was adopted in the FE model.



(a) Connection T1 (b) Connection T2

Fig. 4 Configurations of test specimens (Huns 2002)



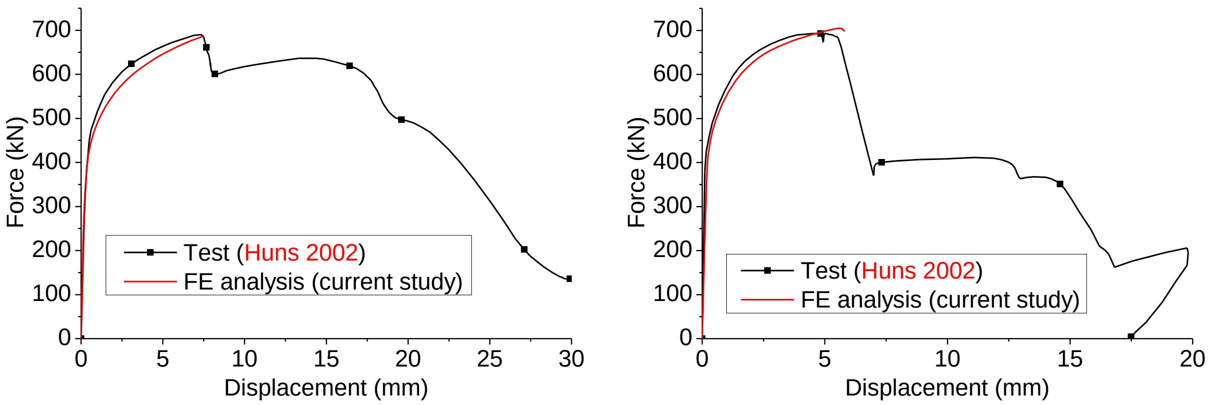
(a) Connection T1 (b) Connection T2

Fig. 5 Finite element models of specimens

4. VALIDATION

To validate the present numerical model, two block shear tests conducted by Huns et al. (2002) were analysed. The configurations of the test specimens are illustrated in Fig. 4. The test gusset plate was connected to the loading devices at both sides by bolts, and one side of it was designed to fail in block shear. The thickness of the gusset plate is 6.6 mm, and the material is grade 350W steel. The diameter of the bolts is 19 mm and that of the bolt holes is 20.6 mm. More details of the tests can be found in Huns et al. (2002).

Because of the symmetry of the gusset plate, only a part of the test gusset plate was modelled, as illustrated in Fig. 4. The finite element models of the modelled zones are shown in Fig. 5. Comparisons between the analysis results and test results are presented in Fig. 6. The displacement shown in Fig. 6 represents the elongation of the tested connection, as illustrated in Fig. 4. As can been seen in Fig. 6, the force-displacement curves obtained from the current analysis match well with test results. The difference of the ultimate strength between the analysis and the test results is around 1% and 2% for connections T1 and T2, respectively. Since static analysis method was adopted, the behaviour of the connection after the ultimate load was not traced.



(a) Connection T1 (b) Connection T2

Fig. 6 Comparison of analysis results and test results

5. RESULTS

The proposed test specimens presented in Section 2 above were simulated using the validated FE models. Same modelling technique was used for the analysis of the proposed test specimens and that in the validation section. The FE analysis results of the test specimens are presented in the following sections.

5.1 Failure mechanism

The analysis results show that all the 6 proposed angle specimens were failed in block shear with a combination of rupture of the tension plane and yielding of the shear plane. Specimen B-Bt2 is used to illustrate the failure mechanism of block shear. As shown in Fig. 7, element removal occurred near the bolt hole edge at the tension plane, indicating tensile rupture was initiated. It can also be seen that the maximum shear stress in the shear plane is 487.7 MPa, which is 61.6% of the nominal tensile strength of steel Q690. Since the nominal shear stresses of yield and ultimate strength of a metal material are 57.7% of the tensile yield strength and ultimate strength of it respectively, the shear plane was not only yielded but also reached the nominal ultimate shear strength.

5.2 Effects of material grade

Fig. 8 compares the force-displacement curves of angles with the same geometrical dimensions but different steel grades, in which the displacement is the relative displacement between the left end of gusset plate and the symmetry plane as illustrated in Fig. 2. The figure shows that the angle specimens of steel S275 failed at a larger displacement than those of angles of steel Q690 and Q960, while the difference between the latter two is small. This means that the angles of S275 steel failed with much higher ductility. The shear stress (σ_{12}) in the shear plane of the above angles at the time of reaching the ultimate loads are summarized in Table 2. It can be seen from the table that for the angle specimens of steel grade S275, the shear stress at the ultimate load level is larger than the yield strength f_y. However, the Q690 and Q960 angles could only reach 65% and 63% of the yield strength of the corresponding materials, respectively. The ratios of shear stress and tensile strength of the three steel grades as shown in the table are around 0.70, 0.62 and 0.61 for steel S275, Q690 and Q960, respectively. Although the ductility of steel Q690 and Q960 is lower than that of steel S275, the maximum shear stress in the shear plane of angle can still reach around 0.6 of the tensile strength of the material.

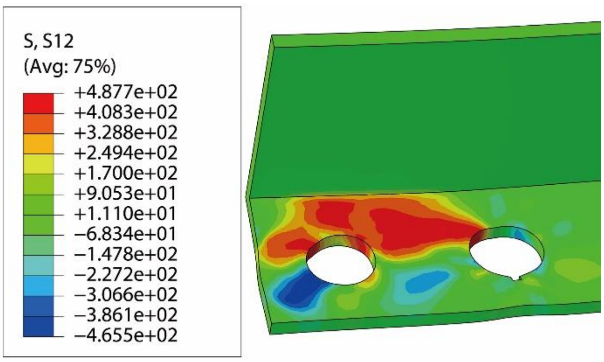


Fig. 7 Failure of specimen B-Bt2-e2-p2

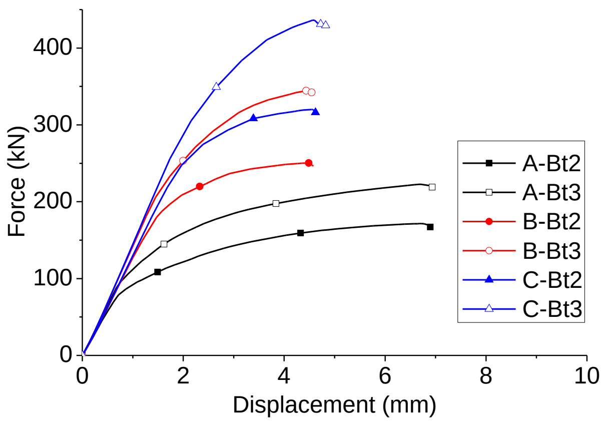


Fig. 8 Response curves of specimens

Table 2 Shear stress of specimens at the time of maximum load

Specimen	f_y (MPa)	f_u (MPa)	f_u/f_y	σ_{12} (MPa)	σ_{12}/f_y	σ_{12}/f_u
A-Bt2	338.5	530.6	1.57	371.8	1.10	0.70
A-Bt3	338.5	530.6	1.57	360.9	1.07	0.68
B-Bt2	750.4	792.0	1.06	491.1	0.65	0.62
B-Bt3	750.4	792.0	1.06	486.6	0.65	0.61
C-Bt2	997.9	1038.2	1.04	628.9	0.63	0.61
C-Bt3	997.9	1038.2	1.04	625.2	0.63	0.60

5.3 Comparisons with design code equations

This section compares the analysis results with the predictions according to the design equations stipulated in AISC-2010 (2010), Eurocode 3 (2005) and the Canadian standard CSA S16-09 (2009). Safety factors are not considered in the design equations for comparison purpose. According to AISC-2010, the block shear strength can be calculated according to the following equation:

$$R_n = U_{bs} f_u A_{nt} + 0.60 f_u A_{nv} \leq U_{bs} f_u A_{nt} + 0.60 f_y A_{gv} \tag{1}$$

where R_n is block shear strength; U_{bs}=1.0; A_{nv}, A_{nt} and A_{gv} are illustrated in Fig. 9.

The block shear strength according to Eurocode 3 is as follows:

$$R_n = f_u A_{nt} + \left(\frac{1}{\sqrt{3}}\right) f_y A_{nv} \tag{2}$$

The equation of block shear strength in CSA S16-09 is:

$$R_n = U_t f_u A_{nt} + 0.6 A_{gv}(f_y + f_u)/2 \tag{3}$$

where $U_t = 0.6$ for angle.

Comparison of the analysis results and standard predictions about block shear strength are summarized in Table 3. It can be seen from this table that AISC-2010 and Eurocode 3 give conservative predictions of the block shear strength of angles irrespective of the steel grades examined, and Eurocode 3 gives more conservative results than AISC-2010. On the other hand, CSA S16-09 overestimates the strength of all the angles of Q690 steel and Q960 steel. It is worth noting that CSA S16-09 overestimates the block shear strength of specimen A-Bt2 but underestimates that of specimen A-Bt3, although both specimens have the same steel grade of S275.

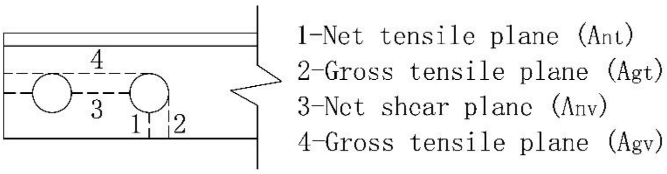


Fig. 9 Illustration of calculating planes

Table 3 Analysis results and standard predictions of block shear capacity

Specimen	P_u (kN)	P_{code}/P_u		
		AISC-2010	Eurocode 3	CSA S16-09
A-Bt2	171.560	0.79	0.60	0.91
A-Bt3	222.740	0.83	0.60	1.05
B-Bt2	250.293	0.80	0.76	1.05
B-Bt3	344.454	0.80	0.75	1.15
C-Bt2	320.070	0.83	0.79	1.08
C-Bt3	436.257	0.83	0.78	1.21

6. CONCLUSIONS

This paper presents a numerical study of the block shear strength of bolted angles of steel grade S275, Q690 and Q960. The comparisons between the analysis results and the predictions by current design standards are also conducted to evaluate their applicability to predict the block shear strength of HSS angles, especially those of grade Q690 and Q960 steels. The following key conclusions are noted:

(a) The ductility of bolted angles of steel Q690 and Q960 are lower than that of angles of steel S275.
(b) Bolted angels of steel Q690 and Q960 could fail in block shear, and have sufficient ductility to enable the shear stress in the shear plane to reach 60% of the nominal tensile strength of the material at the time of failure.
(c) The design standards AISC-2010 and Eurocode 3 give conservative predictions of the block shear strength of bolted angles for both normal and HSS steel of Q690 and Q960. The Canadian standard CSA S16-09 overestimates the strength of angles of Q690 steel and Q960 steel, while its predictions of block shear strength of angles of S275 steel could be either conservative or unsafe.

ACKNOWLEDGEMENT

The work in this paper is fully supported by a grant from the Research Grants Council of the Hong Kong Special Administrative Region, China (Project No. PolyU 152650/16E).

REFERENCES

ABAQUS (2014), “ABAQUS User's Manual”, Version 6.14-4.
American Institute of Steel Construction. (2010), “ANSI/AISC360-10: Specification for structural steel buildings”. Chicago, America.
Chesson, E. (1959), “Behavior of large riveted and bolted structural connections”, PhD thesis, University of Illinois Urbana: Illinois.
Clements, D.D.A. and Teh, L.H. (2013), “Active shear planes of bolted connections failing in block shear”, Journal of structural engineering, 139(3): p. 320 - 327.
Clements, D.D.A. and Teh, L.H. (2012), “Block Shear Capacity of Bolted Connections in Cold-Reduced Steel Sheets”, Journal of structural engineering, 138(4): p. 459 - 467.
Canadian Standards Association (2009), “CSA S16-09: Limit states design of steel structures”. Toronto, Canada.
Epstein, H.I. (1992), “An experimental-study of block shear failure of angles in tension”, Engineering journal-American institute of steel construction inc, 29(2): p. 75-84.
European Committee for Standardization (2005), “EN 1993-1-8: Design of steel structures, Part 1.8: Design of joints”. Brussels, Belgium.
Gross, J., Orbison, J. and Ziemian, R. (1995), “Block shear tests in high-strength steel angles”, Engineering journal-American institute of steel construction inc, 32: p. 117-122.

Huns, B.B.S., Grondin, G.Y. and Driver, R.G. (2002), "Block shear behaviour of bolted gusset plates", Report, Department of civil and environmental engineering, University of Alberta.

Kim, H.J. and Yura, J.A. (1999), "The effect of ultimate-to-yield ratio on the bearing strength of bolted connections", Journal of constructional steel research, 49(3): p. 255 - 269.

Kouhi, J. (1992), "Strength tests on bolted connections using high strength steels (HSS steels) as a base material", Rakenteiden mekaniikka, 25(3): p. 41-53.

Kulak, G.L. and Grondin, G.Y. (2001), "AISC LRFD rules for block shear in bolted connections - A review", Engineering journal-American institute of steel construction inc, 38(4): p. 199-203.

Ling, T.W., Zhao, X.L., Al-Mahaidi, R. and Packer, J.A. (2007), "Investigation of block shear tear-out failure in gusset-plate welded connections in structural steel hollow sections and very high strength tubes", Engineering structures, 29(4): p. 469-482.

Topkaya, C. (2004), "A finite element parametric study on block shear failure of steel tension members", Journal of constructional steel research, 60(11): p. 1615-1635.