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Block shear failure planes of bolted connections — Direct experimental verifications

Lip H. Teh *, Mehmet E. Uz

School of Civil, Mining & Environmental Engineering, University of Wollongong, Wollongong, NSW 2522, Australia

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ABSTRACT

This paper presents direct experimental verifications of the active shear planes in bolted connections, previously identified by the first author for determining the block shear capacity. The laboratory test results were obtained by independent researchers for specimens where the applied loads were resisted by the "block" in shear only. The first set consists of five bolted connection specimens in the webs of wide flange sections where the tensile resistance planes had been sawn off. The second set consists of ten bolted connection specimens each in one leg of an angle section that had fractured completely along the net tensile plane through a block shear failure. Comparisons among the gross, net, and active shear planes against the independent laboratory test results showed that the critical shear planes of bolted connections were best represented by the active shear planes rather than either the gross or the net shear planes. It is also pointed out that full or almost full shear strain hardening was generally achieved at the ultimate limit state of block shear failure of bolted connections in hot-rolled steel plates or sections, irrespective of the connection length. Verification against independent laboratory test results of tee sections bolted at the web reinforces this point.

1. Introduction

Teh and Clements [1] have described how the design provisions against block shear failures of bolted connections [2–7] oscillated over the years since its discovery in 1978 by Birkemoe and Gilmore [8], as summarised in Table 1. The most important factor causing the uncertainty was the use of the gross and the net areas in computing the yielding and the rupture resistance terms, respectively. The gross shear area, annotated in Fig. 1(a) as A_{gv}, is used when the failure mechanism is shear yielding and tensile rupture [3,5,7], while the net shear area, annotated in Fig. 1(b) as A_{nv}, is used for the shear rupture and tensile yielding mechanism [3,6] or simultaneous shear and tensile rupture mechanism [2,4,7]. The inconsistent definitions for the failure planes gave rise to unnecessary anomalies that led to repeated amendments to the design provision.

In a recent paper, Clements and Teh [9] have argued that, whether it is shear yielding or shear rupture, the critical shear planes lie midway between the gross and the net shear planes, as indicated by the results of nonlinear contact finite element analysis. These shear planes, depicted in Fig. 2 and denoted A_{av}, are termed the active shear planes [1,9] to distinguish them from the gross and net shear planes. In Fig. 2, the variable n_r is the number of bolt rows, d_h is the bolt hole diameter, and t is the plate thickness.

Although the finite element analysis results of Clements and Teh [9] clearly showed the location of the active shear planes, there has been no direct experimental verification supporting the use of the active shear planes. Comparisons among the gross, net, and active shear planes against laboratory test results were carried out in the context of block shear failures that invariably involved both tensile and shear resistance components [1,9]. A more convincing case will result if the comparisons are made against laboratory test specimens each of whose ultimate load depends on the shear resistance only. In addition, verifications against independent laboratory test results obtained by different researchers are desirable.

In this paper, comparisons among the gross, net, and active shear planes will be made against independent laboratory test results where only the shear resistance is relevant. Five of the specimens are bolted connections in the webs of wide flange sections where the tensile resistance planes had been sawn off so that the applied loads were resisted by shear only [10], as illustrated in Fig. 3(a). The other ten specimens are bolted connections each in one leg of an angle section that had fractured completely along the net tensile plane through a block shear failure [11], as illustrated in Fig. 3(b). As the stroke displacement was continued following the tensile fracture, the applied load was resisted by shear only.

This paper also includes discussions on strain hardening during shear yielding leading to the block shear failure of a bolted connection, and verification of the proposed block shear equation against tee sections bolted at the web. Although the block shear failure mode was

* Corresponding author. Tel.: +61 242213564, fax: +61 242213238.
E-mail address: lteh@uow.edu.au (L.H. Teh).

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Table 1
AISC's block shear design equations over the years.

Version	Block shear provision
1978	$R_n = F_u A_{nt} + 0.6 F_u A_{nv}$
1986	$R_n = \max(F_u A_{nt} + 0.6 F_y A_{gv};\quad F_y A_{gt} + 0.6 F_u A_{nv})$
1989	$R_n = F_u A_{nt} + 0.6 F_u A_{nv}$
1993	If $F_u A_{nt} \geq 0.6 F_u A_{nv} : R_n = F_u A_{nt} + 0.6 F_y A_{gv}$
	If $F_u A_{nt} \leq 0.6 F_u A_{nv} : R_n = F_y A_{gt} + 0.6 F_u A_{nv}$
1999	If $F_u A_{nt} \geq 0.6 F_u A_{nv} : R_n = \min(F_u A_{nt} + 0.6 F_u A_{nv};\quad F_u A_{nt} + 0.6 F_y A_{gv})$
	If $F_u A_{nt} \leq 0.6 F_u A_{nv} : R_n = \min(F_u A_{nt} + 0.6 F_u A_{nv};\quad F_y A_{gt} + 0.6 F_u A_{nv})$
2005	$R_n = \min(F_u A_{nt} + 0.6 F_u A_{nv};\quad F_u A_{nt} + 0.6 F_y A_{gv})$
2010	

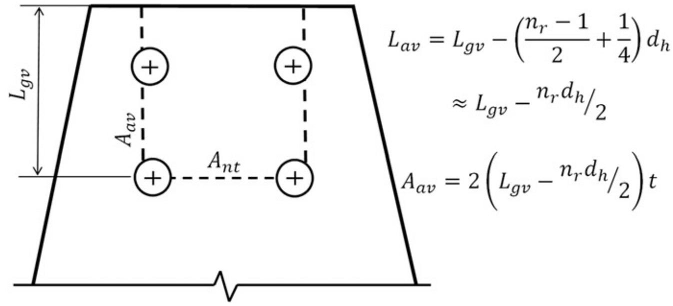

Fig. 2. Active shear planes.

discovered thirty five years ago [8], there is continuing research in this area as represented by very recent papers [12–15].

2. Shear resistance equations

The net shear area A_{nv} depicted in Fig. 1(b) has always been used in the North American specifications [2–7,16] for determining the shear rupture resistance,

$$P_v = 0.6 F_u A_{nv} \tag{1}$$

although it is also used in the Eurocode [17] for determining the shear yield resistance, where the tensile strength F_u is replaced with the yield stress F_y. In any case, Clements and Teh [9] have pointed out that the approach of using the net shear area A_{nv} ignores the fact that the planes coinciding with the centrelines of the bolt holes in the direction of loading do not have maximum shear stresses due to the bolt bearing condition.

Gross et al. [11] suggested that, based on their laboratory test results involving bolted connections that had fractured completely along the net tensile planes and were subjected to continued loading, the critical shear area was the gross shear area A_{gv} depicted in Fig. 1(a). When the net shear area A_{nv} was used for computing the shear rupture resistance, the average shear coefficient was found to be 0.71, significantly higher than the accepted value of 0.6. Therefore, according to Gross et al. [11], the shear rupture resistance would be computed from

$$P_v = 0.6 F_u A_{gv}. \tag{2}$$

However, Gross et al. [11] did not verify their suggestion against their laboratory test results. Eq. (2) is therefore verified in the next section of this paper.

The finite element analysis results of Clements and Teh [9] have indicated that the shear rupture resistance should be computed from the active shear area A_{av} depicted in Fig. 2,

$$P_v = 0.6 F_u A_{av}. \tag{3}$$

3. Verification of active shear planes against laboratory test results

Orbison et al. [10] sawed off the net tensile plane of their bolted webs, as illustrated in Fig. 3(a), so that the applied load of each bolted connection specimen was resisted in the web by shear only. The ultimate test load is denoted *as* P_t in the present work. All specimens were composed of A36 steel W12 × 14 sections, which had a nominal web thickness t of 5.26 mm. The A490 bolts used in the laboratory tests had a nominal diameter of 19 mm, with a bolt hole diameter of 20.6 mm as indicated in Fig. 3(a). The bolt spacing in the loading direction was invariably 63.5 mm. The only relevant (nominal) geometry variables for their specimens were the number of bolt rows n_r and the end distance e, which are given in Table 2 for each specimen.

It can be seen from the ratios of ultimate test load to predicted shear resistance P_t/P_v, termed the professional factor in the literature [10,11,19], in Table 2, that Eq. (3), which is based on the active shear planes, provides the most accurate estimates for the shear resistance. For every specimen, the professional factor of Eq. (3) is considerably closer to unity compared to the other two equations, which make use of the net and gross shear areas respectively.

Gross et al. [11] continued the stroke displacement after an angle specimen bolted at one leg had failed in block shear, and the net tensile plane had fractured completely. In such a case, the applied load was resisted by the "block" in shear only, as illustrated in Fig. 3(b). Ten A588 Grade 50 and three A36 steel angle braces, all with a nominal thickness of 6.35 mm, were tested in this manner. However, three of the A588 specimens failed in a different mode and their results were

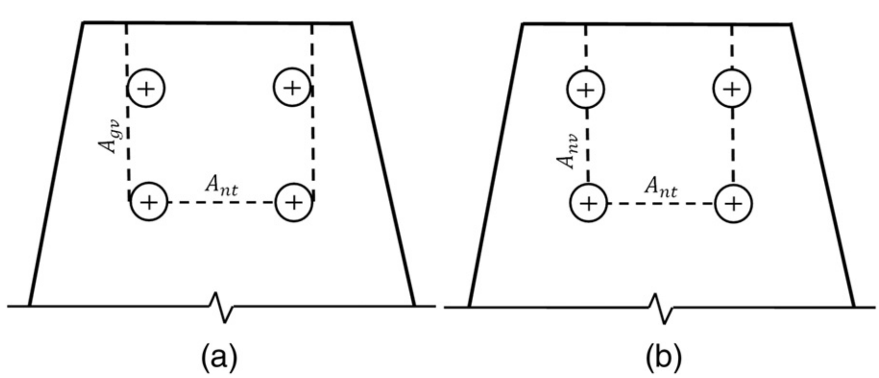

Fig. 1. Nominal shear planes: (a) gross; (b) net.