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Whitmore Tension Section and Block Shear

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Keywords
shear, block, whitmore, tension, section

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The Whitmore Tension Section and Block Shear

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Abstract:

This paper examines the validity of the Whitmore net section tension capacity for the design of bolted gusset plates. Using simple algebra, this paper first shows that the Whitmore criterion and the correct block shear criterion would give similar results for a standard connection having approximately seven rows of bolts. It then shows that the Whitmore criterion severely underestimates the actual capacities of connections having two or three bolt rows tested by independent researchers. Conversely, it also shows that the same criterion overestimates the capacities of connections having nine bolt rows that were believed by the testing researchers to fail in the Whitmore section. Using finite element analysis incorporating fracture simulation, this paper shows that the apparent Whitmore tensile fractures only took place because the tests were continued long after the ultimate limit state of block shear. This paper proposes that the Whitmore section check be made redundant in light of the block shear check, which accurately predicted the ultimate test loads of all the specimens.

Author keywords: bolted connection, block shear, connection capacity, connection design, gusset plate, steel connection, Whitmore section

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Introduction

In Section J4.1 “Strength of Elements in Tension” of the Specification for Structural Steel Buildings (AISC 2016), the effective net area $A_e$ of a connection plate may be limited to that calculated using the Whitmore section. The Whitmore section is defined by drawing 30° lines from the pair of outer downstream bolts to their respective intersections with a line passing through the upstream row of bolts, as illustrated in Figure 1(a) for a bolted gusset plate. The Whitmore section was described by Whitmore (1952) as a reasonable method for approximating the maximum tensile (and compressive) elastic stress incurred in a riveted gusset plate by the axial force in a connected brace, and found widespread use from the late 1970s (Thornton & Lini 2011). However, the Whitmore net section tension capacity was not explicitly mentioned in the AISC specifications until the 2010 edition (AISC 2010) in the form of User Note in Section J4.1.

Incidentally, around the time the Whitmore section became widely known among the structural engineering community, Birkemore & Gilmor (1978) discovered the block shear failure mode for coped beam shear connections. The occurrence of this failure mode in bolted gusset plates and braces were subsequently studied by Richard et al. (1983), Hardash & Bjorhovde (1985), Gross & Cheok (1988), Cunningham et al. (1995), Aalberg & Larsen (1999), Menzemer et al. (1999) and Topkaya (2004) among many others. Studies involving both the Whitmore net section and the block shear failure modes of bolted gusset plates were conducted in recent years by Higgins et al. (2010), Liao et al. (2011) and Rosenstrauch et al. (2013).

According to Kulak et al. (2001, pg 253), the design of a gusset plate is to be checked against both the Whitmore section and the block shear failure mode, and the more severe requirement resulting from them should then be applied. Similarly, a reviewer of a recent paper (Teh & Deierlein 2017) argued that the design example presented therein was controlled by the Whitmore tension rupture, and the block shear criterion was therefore irrelevant to the example.

However, the authors have not found any convincing experimental evidence indicating the failure mode of a bolted gusset plate that corresponds to the Whitmore net section, which would involve simultaneous (or near simultaneous) fractures on both sides of each bolt hole, as illustrated in Figure 1(b). It should be noted that, in cases where fracture can be observed to have taken place on the outer side of the bolt hole(s), the prevailing failure mode was actually block shear rather than Whitmore section fracture, as demonstrated later in this paper. The outer side of the bolt hole only fractured...
when testing continued well beyond the ultimate (block shear) limit state of the specimen, after the inner region fractured completely. Such cases include Specimen 28 tested by Hardash & Bjorhovde (1985), who correctly identified the failure mode to be block shear.

Bjorhovde & Chakrabarti (1985) presented laboratory test results of bolted gusset plate connections, and concluded that their test results were in acceptable agreement with the Whitmore concept. Rabern (1983) had earlier noted that his finite element stress contours suggested that a block shear criterion, modified from that derived for a coped beam (Birkemoe and Gilmor 1978), might be applicable to a bracing connection of a gusset plate. However, Rabern (1983) concluded that his finite element studies supported the Whitmore criterion for gusset plate design. While this conclusion was affirmed by Richard et al. (1983), they also stated that the block shear concept might lead to a more compact and efficient connection.

The ambiguity regarding the finite element finding as described in the preceding paragraph can also be found in the work of Williams (1988), cited by Williams & Richard (1996). However, Williams & Richard (1996) argued that the block shear and the Whitmore criteria gave the same result. This argument is consistent with the statement of Gross & Cheok (1988) that the Whitmore design criterion was essentially the same as the block shear design concept.

Astaneh-Asl (1998), on the other hand, made a distinction between “fracture along the Whitmore’s 30-degree effective width” and “block shear failure”, both of which were claimed to have been observed in the field after earthquakes or in laboratories.

More recently, Rosenstrauch et al. (2013) stated that the so-called direct tension method (FHWA 2009), which evaluates the yield or ultimate capacity of the Whitmore section, did not accurately predict the onset of plasticity or the behaviour of gusset plates. They argued that their finite element analysis showed that block shear and the so-called global section shear were possible failure mechanisms for gusset plates in bridge connections. The guidance document issued by the Federal Highway Administration (FHWA 2009) was meant to assist the rating process of bridges in the wake of the 2007 I-35W Bridge collapse in Minneapolis, Minnesota. The document was believed by some parties to yield overly conservative gusset plate ratings (AASHTO 2013, NCHRP 2013).

It is also noteworthy that the application of the Whitmore section for tension failures has often led to confusions in practice when the effective width crosses a connected edge, as illustrated in Figure 2.
for the gusset plate where the Whitmore effective width runs into the horizontal member. More potential confusions have been described by Thornton & Lini (2011).

This paper aims to establish that the Whitmore criterion for the design of a bolted gusset plate under tension is redundant provided the correct block shear check is performed. The Whitmore tension capacity will first be compared algebraically against the block shear equation to articulate their numerical relationship in terms of the connection geometry. Estimates of the ultimate test loads for laboratory specimens given by the Whitmore and the block shear criteria will then be verified against the laboratory test results. The specimens include those for which the Whitmore tension capacity is much lower than the block shear capacity, and those that were considered by the testing researchers to have failed by tension in the Whitmore net section.

Finite element analysis including fracture propagation will be presented to show that the fractures across the Whitmore net section only took place long after the ultimate limit state of block shear had passed, and well after the complete fracture of the net section within the block shear zone. In addition, it will be demonstrated that the ultimate load of a bolted gusset plate failing in block shear is often reached due to necking of the net tensile section, before the occurrence of fracture.

Additional supporting test data and analysis results are provided in the last three tables pursuant to the comments of reviewers of the original manuscript. This paper is not concerned with the Whitmore effective width for the design against buckling of a gusset plate under compression, which has been shown to be grossly inaccurate (Cheng et al. 2000, Sheng et al. 2002). Astaneh-Asl (1998) has also suggested that the Whitmore concept was intended for gusset plates in tension only.

**Comparisons between the Whitmore section and the block shear mode**

According to Equation (J4-2) of the specification (AISC 2016), the Whitmore tension capacity of the bolted gusset plate in Figure 1(a) is equal to

\[
R_s = F_u A_e = F_u W_w t = F_u \left[ \left(n_l - 1 \right)\left( g - d_h \right) + \left(2(n_r - 1)\right) p \tan 30^\circ - d_h \right] t
\]

(1)

in which \(F_u\) is the material tensile strength, \(A_e\) is the effective net area, \(W_w\) is the Whitmore net width, \(t\) is the plate thickness, \(n_l\) is the number of bolt lines in the direction of loading (equal to 2 in Figure 1), \(g\) is the gauge, \(d_h\) is the bolt hole diameter, \(n_r\) is the number of bolt rows perpendicular to the loading direction (equal to 4 in Figure 1), and \(p\) is the pitch.
Teh & Deierlein (2017) have provided the following block shear capacity

\[ R_n = F_u A_{nt} + 0.6 F_u A_{ev} \]

\[ = F_u \left[ (n_t - 1)(g - d_h) + 1.2 \left( (n_r - 1)p + (2n_r - 1)/4 d_h \right) \right] t \]  

(2)

in which \( e_1 \) is the end distance defined in Figure 3. The tensile and shear resistance planes in the block shear mode are indicated in the figure. It should be noted that the shear resistance area \( A_{ev} \) is the mean between the gross and the net shear areas defined in the specification (AISC 2016).

Equation (2) has been demonstrated by Teh & Deierlein (2017) to be accurate and reliable through verifications against 161 bolted gusset plate specimens tested by independent researchers around the world. The first term in the first line of Equation (2) represents the tensile resistance, while the second term is the shear resistance of the block. It should be noted that the use of the material strength \( F_u \) in the shear resistance term does not account for shear fracture, but for shear yielding at full strain hardening (Teh & Uz 2015).

From Equations (1) and (2), it can be derived that if the pitch \( p \) is equal to three times the bolt hole diameter \( d_h \), and the end distance \( e_1 \) is 1.5 times \( d_h \), then the Whitmore tension capacity will be equal to the block shear capacity when the number of bolt rows \( n_r \) is equal to 6.7:

\[ 2(n_r - 1)(3d_h) \tan 30^\circ - d_h = 1.2 \left( (n_r - 1)(3d_h) + (1.5d_h) - \left( (2n_r - 1)/4 d_h \right) \right) \]

\[ n_r = 6.7 \]

(3)

Therefore, the ultimate capacity of a typical gusset plate connection with approximately seven rows of bolts may be accurately estimated using the Whitmore criterion even if it fails in block shear, giving the false impression that the Whitmore tension section were valid.

If the connection has only a few rows of bolts, then the Whitmore tension capacity will be significantly lower than the block shear capacity. This algebraic outcome enables the verification of the Whitmore criterion against the laboratory test results of such specimens.

Table 1 shows the results of Equations (1) and (2) for the specimens tested by Aalberg & Larsen (1999), which were known to have failed in block shear. The variable \( F_y \) is the (measured) yield stress of the steel material, given here for the purpose of the finite element analysis discussed in the next section. The variable \( P_t \) denotes the ultimate load obtained in the laboratory test, and the ratio
"P/Rn is known as the professional factor. An empty cell in the following tables indicates that the entry in the cell above applies.

It can be seen from Table 1 that the ultimate test loads of the specimens with two rows of bolts were up to 90% higher than the computed Whitmore tension capacity. The extent of underestimation decreases with increasing number of bolt rows. In any case, it is clear that, had the Whitmore tension fracture mode existed, the specimens would have failed at loads significantly lower than their actual ultimate loads. The results presented in Table 1 is an unambiguous indication that the Whitmore tension capacity does not exist.

Bjorhovde & Chakrabarti (1985) presented laboratory test results of bolted gusset plates that were believed to have failed in the Whitmore section. Photographs of two specimens seem to indicate fractures in the Whitmore zone similar to that illustrated in Figure 1(b), i.e. the tension fracture extends into the outer Whitmore zone beyond the block shear perimeter indicated in Figure 3. However, there are two points worth noting regarding this apparent indication. First, the number of bolt rows \( n \) in each specimen is equal to 9, so the Whitmore tension capacity is greater than the block shear capacity. It will therefore be instructive to compare the estimates of Equations (1) and (2) against each other, knowing that the latter governs (in contrast to Table 1). Second, it was not clear whether the fractures in the outer Whitmore zone took place before the inner zone (the block shear zone) completely fractured, or after it. The first point is investigated in this section, while the second in the next.

Table 2 shows that the professional factors of Equation (2) are noticeably closer to unity compared to Equation (1) for both specimens while all of them are less than or equal to unity, suggesting that the specimens failed in block shear. It is therefore quite possible that the fractures in the outer Whitmore zone took place after the tests were continued well beyond the respective block shear failures, associated with fractures in the net tensile section of the block.

**Finite element investigations**

Finite element analysis is an excellent tool to investigate the tensile stress contours at the ultimate limit state of a bolted gusset plate, and to corroborate the inference made in the preceding paragraph. The use of the hexahedral reduced integration brick element C3D8R available in ABAQUS 6.12 Standard (ABAQUS 2012) also enables the demonstration that the ultimate load is due to (out-of-plane) necking in the net tensile section, not fracture. It should be noted that this phenomenon and
the associated gradual drop in resistance cannot be captured by the use of 2D elements such as that employed by Wen & Mahmoud (2017).

In order to reduce the analysis time and minimise possible numerical precision errors, advantage was taken of the symmetry of the bolted gusset plates studied in the present work. The modelling of symmetry conditions for such plates has been described by Clements & Teh (2013). Furthermore, each of the bolts was modelled as a 3D analytical rigid body revolved shell as their deformations had little effects on the gusset plate's failure mode. The bolts were displaced together to simulate loading of the gusset plate as the displacement would be resisted by the surface contact between the bolt and the bolt hole at the downstream end, in the same manner as conducted by Clements & Teh (2013).

However, unlike the work of Clements & Teh (2013), fracture initiation and propagation were simulated in the present work in order to investigate the Whitmore section fracture postulated in Figure 1(b) and apparently found by researchers in the literature. The simulations using ABAQUS/Standard also enable the development sequence of fractures and their relationships to the resistance level of the gusset plate be studied. Element deletion was activated when the maximum degradation was reached at an integration point.

The present finite element models are validated in Table 1, where it can be seen that there are excellent agreements in the ultimate loads between the test results and the FEA results. Modelling of the stress-strain curve and the damage parameters of a certain specimen is described in the following subsection, where validation of the load-deflection graph can also be found.

**Specimen underestimated by the Whitmore criterion**

As shown in Table 1, the ultimate test load of Specimen T-16 tested by Aalberg & Larsen (1999) was 27% higher than the computed Whitmore tension capacity. If the underestimation was simply a numerical inaccuracy, then there would have been fractures in the outer Whitmore zone as illustrated in Figure 1(b). However, such fractures are not evident in the photograph of the tested specimen provided by Aalberg & Larsen (1999), even though the test was continued until the shear planes fractured. The conditions of the specimen at the ultimate limit state and beyond are studied using the present finite element analysis.
The true stress-strain curve used in the analysis is plotted in Figure 4. Up to the horizontal portion, the engineering stress-strain relationship was first defined using the Ramberg-Osgood power model (Ramberg and Osgood 1943)

\[ \varepsilon = \frac{\sigma}{E} + 0.002 \left(\frac{\sigma}{F_y}\right)^n \]  

(4)

in which \( \varepsilon \) is the engineering strain, \( \sigma \) is the engineering stress and \( E \) is the Young’s modulus of elasticity. The power term \( n \) is determined from

\[ n = \frac{\ln\left(\varepsilon_{us}/0.2\right)}{\ln\left(F_{us}/F_y\right)} \]  

(5)

in which \( \varepsilon_{us} \) is defined as

\[ \varepsilon_{us} = 100\left[\varepsilon_u - \left(F_{us}/E\right)\right] \]  

(6)

The variable \( \varepsilon_u \) is the engineering strain at the ultimate stress. For Specimen T-16, it is assumed to be 10%.

Having defined the engineering stress-strain relationship as given by Equation (4), the true stress-strain curve (up to the horizontal portion shown in Figure 4) was plotted from

\[ \varepsilon_{true} = \ln\left[1 + \varepsilon\right] \]  

(7)

and

\[ \sigma_{true} = \sigma\left[1 + \varepsilon\right] \]  

(8)

The damage initiation parameters used in the present work are shown in Table 3, which have been obtained by trial and error to reasonably match the response of the spliced (bolted) tension coupon tested by Aalberg & Larsen (1999) and depicted in Figure 5(a). The equivalent plastic displacement at failure is set to be 0.2. The calibration result is shown in Figure 5(b). It should be noted that the elastic portion (initial stiffness) of the experimental curve has been adjusted to account for the fact that there were small misalignments in the bolted coupon as no attempt was made by Aalberg & Larsen (1999) to achieve perfect alignment of the bolt holes during the experiment. The plasticity of the steel material was handled through the von Mises yield criterion and the Prandtl-Reuss flow rule.
with isotropic hardening. The elastic modulus is assumed to be 200 GPa, and the Poisson’s ratio is 0.3.

In the following Figures 6 through 8 for Specimen T-16, the mirror images of the symmetric-half FEA models are added to facilitate illustration. Figure 6(a) shows the out-of-plane necking of the net tensile section within the block shear zone at the ultimate limit state, which occurs at a simulated load of 956 kN, or 0.5% lower than the ultimate test load obtained by Aalberg & Larsen (1999). It can be seen from the out-of-plane displacement contours that necking is confined between the two bolt holes only, not extending into the outer Whitmore zone despite the applied load being 27% higher than the Whitmore capacity. The corresponding longitudinal normal stress contours are shown in Figure 6(b).

Shear displacement of the “block” is also evident in Figure 6. The existence of the block is indicated by the von Mises stress contours in Figure 7. It can be seen that the von Mises stresses around the block shear perimeter are significantly higher than in the outer Whitmore zone, vindicating the use of the block shear criterion rather than the Whitmore criterion despite the latter’s lower capacity. Shear displacement of the block becomes more pronounced following the net tensile section rupture, as shown in Figure 8(a).

Figure 8(b) shows that, even after the net tensile section of the block fractured completely, and the block continued to shear, there is no evidence of necking or fracture in the outer Whitmore zone.

The load-deflection graph obtained by the finite element analysis can be compared to that obtained by Aalberg & Larsen (1999) in Figure 9. States corresponding to Figures 6, 8(a) and 8(b) are annotated along the curve.

As indicated in Figure 9, the ultimate block shear load is due to necking of the net tensile section, not due to fracture. This point has been previously explained by Teh & Clements (2012). In fact, fracture only took place after extended gradual softening of the response under quasi-static loading of the high-strength steel specimens, as annotated in the figure.

*Specimen showing apparent Whitmore fracture*

As mentioned in the section “Comparisons between the Whitmore section and the block shear mode”, two nine-row bolted gusset plates of Bjorhovde & Chakrabarti (1985) were shown in
photographs (see Figure 10 for example) to fracture in the outer Whitmore zone. However, Table 2 shows that the ultimate test loads of both specimens, which were nominally identical to each other, were closer to the block shear capacity given by Equation (2) than to the Whitmore tension capacity. It should also be noted that all the computed capacities were on the same side of (un)conservatism. The development of fractures in the specimens is studied in the present finite element analysis, which incidentally models the gusset plate having its loading direction inclined at 45 degrees to the adjacent member.

The true stress-strain curve used in the finite element analysis is plotted in Figure 11, employing the procedure expressed by Equations (4) through (8) based on the assumption that the engineering strain $\varepsilon_u$ of the mild steel is 40%. Since Bjorhovde & Chakrabarti (1985) did not provide any coupon test results, the damage initiation and damage evolution parameters obtained from the preceding calibration against the test results of Aalberg & Larsen (1999) were used in the present analysis.

The simulated ultimate load is 690 kN, or 1.9% lower than the block shear capacity given by Equation (2). Figure 12 shows that the drop from the ultimate load was less gradual compared to that of Specimen T-16 tested by Aalberg & Larsen (1999), shown in Figure 9. However, as shown in Figure 13, the ultimate load taking place at the displacement of 9.3 mm was still due to necking of the net tensile section, although fracture was imminent for the 3.2 mm thick gusset plate with nine rows of bolts.

Figure 14 shows the complete fracture of the net tensile section within the block when the displacement is equal to 15.2 mm, as annotated in Figure 12. Even at this stage, there is no fracture in the outer Whitmore zone. It is only when the displacement reaches 20.8 mm (corresponding to a load of 603 kN) that fracture initiates in the outer Whitmore zone. Figure 15 shows the fracture at a displacement of 24 mm.

The present FEA results, coupled with the comparison results shown in Table 2, clearly indicate that the two gusset plate specimens of Bjorhovde & Chakrabarti (1985) did not fail in the Whitmore section but in block shear.

**Additional test data and modified Whitmore sections**

Tables 4 and 5 compare Equations (1) and (2) for the specimens tested by Huns et al. (2002) and Mullin (2004), respectively. It can be seen that the outcome is consistent with that for Table 1. All
the ultimate test loads of Huns et al. (2002) were more than 25% higher than the Whitmore estimates given by Equation (1), and were significantly closer to the block shear capacities given by Equation (2).

For the specimens of Mullin (2004) which had six to eight rows of bolts, the Whitmore capacities given by Equation (1) are close to the block shear capacities given by Equation (2), supporting the finding of Equation (3) that the two equations will give similar results for connections with approximately seven rows of bolts. For the specimen of Mullin (2004) which had two rows of bolts, the ultimate test load exceeded the Whitmore capacity by 44%, but was only 7% higher than the block shear capacity computed using Equation (2).

Table 6 presents the results of the modified Whitmore sections proposed by Irvan (1957), Chesson & Munse (1963), Yamamoto et al. (1985), Cheng et al. (2000) and Dowswell (2013) for the specimens listed in Table 1. It should be noted that not all of the cited authors necessarily referred to the Whitmore tension section, and were in some cases concerned with the “dispersion angle” under compression. In the case of Irvan (1957), the 30° lines are projected from the geometric centre of the rivet group, resulting in an effective width that is as narrow as one tenth of the Whitmore width. Readers should consult the references for details of the modified Whitmore sections.

In any case, it can be seen from the results given in Table 6 that there is no reliable method for determining the dispersion angle of the Whitmore section, even if the section existed. For some variants, the errors are even more severe than those obtained using the well-established dispersion angle of 30° proposed by Whitmore (1952).

**Conclusions**

The Whitmore criterion has been used by structural engineers to determine the tension capacity of connected steel plate elements. Some authorities suggested or still require that the design of a gusset plate be checked against both the Whitmore and the block shear criteria. Using simple algebra, the paper has shown that the Whitmore criterion only gives a similar result to the (correct) block shear criterion under certain conditions. For a standard bolted connection satisfying the AISC recommendations for the bolt spacing and the end distance, the two criteria would lead to similar results if there are approximately seven rows of bolts.
The Whitmore criterion has been shown in the paper to be excessively conservative for connections having two or three rows of bolts. The ultimate test load obtained by Aalberg & Larsen (1999) for a connection having two rows of bolts was 90% higher than that predicted by the Whitmore criterion. If the Whitmore criterion were valid, such an outcome would not have been possible. The ultimate test loads of the specimens tested by Aalberg & Larsen (1999) were accurately determined using the block shear equation proposed by Teh & Deierlein (2017).

Conversely, for connections having nine rows of bolts tested by Bjorhovde & Chakrabarti (1985), the Whitmore criterion overestimated the ultimate capacities even though the gusset plates were thought by the researchers to have failed in the Whitmore section. The ultimate test loads were closer to the block shear capacity, suggesting that the failure mode was block shear. The actual failure mode has been confirmed through the finite element analysis presented in this paper to be block shear. The finite element analysis has also shown that fractures in the Whitmore zone outside the block only took place because the connection test was continued well beyond the ultimate limit state.

Additional test data and modified Whitmore sections have also been investigated, the results of which confirm the conclusion that the Whitmore section check is not a viable criterion. The paper has demonstrated that the Whitmore section check for the design of a bolted gusset plate under tension is redundant provided the correct block shear check is performed.

By not requiring the Whitmore section check, the design of standard gusset plates having bolt rows less than seven will be more economical. Furthermore, difficulties in applying the Whitmore section check in geometries where the Whitmore section crosses into another member will be obviated.

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Table 1. Comparison of Whitmore and block shear predictions for Aalberg & Larsen (1999)

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*aThese I-section specimens had their flanges removed.

Table 2. Comparison of Whitmore and block shear predictions for Bjorhovde & Chakrabarti (1985)

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<th>$g$ (mm)</th>
<th>$d_h$ (mm)</th>
<th>$t$ (mm)</th>
<th>$n_r$</th>
<th>$n_l$</th>
<th>$F_y$ (MPa)</th>
<th>$F_u$ (MPa)</th>
<th>$P_u/R_n$</th>
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<td>Whitmore Eqn (1)</td>
<td>Block Shear Eqn (2)</td>
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Table 3. Ductile damage parameters

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Table 4. Comparison of Whitmore and block shear predictions for Huns et al. (2002)

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<th>$n_l$</th>
<th>$F_y$ (MPa)</th>
<th>$F_u$ (MPa)</th>
<th>$P_t/R_n$</th>
<th>Whitmore Eqn (1)</th>
<th>Block Shear Eqn (2)</th>
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*While the ultimate test load was cited in some places of the report to be 756 kN, it was given as 691 kN in page 137 of the report. An inspection of the load-deflection graph in page 43 confirms that the lower value is the correct one.

Table 5. Comparison of Whitmore and block shear predictions for Mullin (2002)

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Table 6. Comparison of Whitmore variants and block shear for Aalberg & Larsen (1999)

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