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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.

Keywords

shear, block, whitmore, tension, section

Disciplines

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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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18 Introduction

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

29 Incidentally, around the time the Whitmore section became widely known among the structural
 30 engineering community, Birkemore & Gilmor (1978) discovered the block shear failure mode for
 31 coped beam shear connections. The occurrence of this failure mode in bolted gusset plates and braces
 32 were subsequently studied by Richard et al. (1983), Hardash & Bjorhovde (1985), Gross & Cheok
 33 (1988), Cunningham et al. (1995), Aalberg & Larsen (1999), Menzemer et al. (1999) and Topkaya
 34 (2004) among many others. Studies involving both the Whitmore net section and the block shear
 35 failure modes of bolted gusset plates were conducted in recent years by Higgins et al. (2010), Liao et
 36 al. (2011) and Rosenstrauch et al. (2013).

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

42 However, the authors have not found any convincing experimental evidence indicating the failure
 43 mode of a bolted gusset plate that corresponds to the Whitmore net section, which would involve
 44 simultaneous (or near simultaneous) fractures on both sides of each bolt hole, as illustrated in Figure
 45 1(b). It should be noted that, in cases where fracture can be observed to have taken place on the outer
 46 side of the bolt hole(s), the prevailing failure mode was actually block shear rather than Whitmore
 47 section fracture, as demonstrated later in this paper. The outer side of the bolt hole only fractured

48 when testing continued well beyond the ultimate (block shear) limit state of the specimen, after the
49 inner region fractured completely. Such cases include Specimen 28 tested by Hardash & Bjorhovde
50 (1985), who correctly identified the failure mode to be block shear.

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

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

64 Astaneh-Asl (1998), on the other hand, made a distinction between “fracture along the Whitmore’s
65 30-degree effective width” and “block shear failure”, both of which were claimed to have been
66 observed in the field after earthquakes or in laboratories.

67 More recently, Rosenstrauch et al. (2013) stated that the so-called direct tension method (FHWA
68 2009), which evaluates the yield or ultimate capacity of the Whitmore section, did not accurately
69 predict the onset of plasticity or the behaviour of gusset plates. They argued that their finite element
70 analysis showed that block shear and the so-called global section shear were possible failure
71 mechanisms for gusset plates in bridge connections. The guidance document issued by the Federal
72 Highway Administration (FHWA 2009) was meant to assist the rating process of bridges in the wake
73 of the 2007 I-35W Bridge collapse in Minneapolis, Minnesota. The document was believed by some
74 parties to yield overly conservative gusset plate ratings (AASHTO 2013, NCHRP 2013).

75 It is also noteworthy that the application of the Whitmore section for tension failures has often led to
76 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

$$R_n = F_u A_e = F_u W_w t$$

$$= F_u \left[(n_l - 1)(g - d_h) + \{2(n_r - 1)p \tan 30^\circ - d_h\} \right] t \quad (1)$$

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.

105 Teh & Deierlein (2017) have provided the following block shear capacity

$$\begin{aligned}
 R_n &= F_u A_{nt} + 0.6 F_u A_{ev} \\
 &= F_u \left[(n_l - 1)(g - d_h) + 1.2 \left\{ (n_r - 1)p + e_1 - \frac{(2n_r - 1)}{4} d_h \right\} \right] t
 \end{aligned} \tag{2}$$

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

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

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

$$\begin{aligned}
 2(n_r - 1)(3d_h) \tan 30^\circ - d_h &= 1.2 \left\{ (n_r - 1)(3d_h) + (1.5d_h) - \frac{(2n_r - 1)}{4} d_h \right\} \\
 n_r &= 6.7
 \end{aligned} \tag{3}$$

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

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

126 Table 1 shows the results of Equations (1) and (2) for the specimens tested by Aalberg & Larsen
 127 (1999), which were known to have failed in block shear. The variable F_y is the (measured) yield
 128 stress of the steel material, given here for the purpose of the finite element analysis discussed in the
 129 next section. The variable P_t denotes the ultimate load obtained in the laboratory test, and the ratio

130 P/R_n is known as the professional factor. An empty cell in the following tables indicates that the
 131 entry in the cell above applies.

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

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

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

154 **Finite element investigations**

155 Finite element analysis is an excellent tool to investigate the tensile stress contours at the ultimate
 156 limit state of a bolted gusset plate, and to corroborate the inference made in the preceding paragraph.
 157 The use of the hexahedral reduced integration brick element C3D8R available in ABAQUS 6.12
 158 Standard (ABAQUS 2012) also enables the demonstration that the ultimate load is due to (out-of-
 159 plane) necking in the net tensile section, not fracture. It should be noted that this phenomenon and

160 the associated gradual drop in resistance cannot be captured by the use of 2D elements such as that
 161 employed by Wen & Mahmoud (2017).

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

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

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

179 *Specimen underestimated by the Whitmore criterion*

180 As shown in Table 1, the ultimate test load of Specimen T-16 tested by Aalberg & Larsen (1999) was
 181 27% higher than the computed Whitmore tension capacity. If the underestimation was simply a
 182 numerical inaccuracy, then there would have been fractures in the outer Whitmore zone as illustrated
 183 in Figure 1(b). However, such fractures are not evident in the photograph of the tested specimen
 184 provided by Aalberg & Larsen (1999), even though the test was continued until the shear planes
 185 fractured. The conditions of the specimen at the ultimate limit state and beyond are studied using the
 186 present finite element analysis.

187 The true stress-strain curve used in the analysis is plotted in Figure 4. Up to the horizontal portion,
 188 the engineering stress-strain relationship was first defined using the Ramberg-Osgood power model
 189 (Ramberg and Osgood 1943)

$$190 \quad \varepsilon = \frac{\sigma}{E} + 0.002 \left(\frac{\sigma}{F_y} \right)^n \quad (4)$$

191 in which ε is the engineering strain, σ is the engineering stress and E is the Young's modulus of
 192 elasticity. The power term n is determined from

$$193 \quad n = \frac{\ln(\varepsilon_{us}/0.2)}{\ln(F_u/F_y)} \quad (5)$$

194 in which ε_{us} is defined as

$$195 \quad \varepsilon_{us} = 100 \left[\varepsilon_u - (F_u/E) \right] \quad (6)$$

196 The variable ε_u is the engineering strain at the ultimate stress. For Specimen T-16, it is assumed to be
 197 10%.

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

$$200 \quad \varepsilon_{true} = \ln[1 + \varepsilon] \quad (7)$$

201 and

$$202 \quad \sigma_{true} = \sigma[1 + \varepsilon] \quad (8)$$

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

211 with isotropic hardening. The elastic modulus is assumed to be 200 GPa, and the Poisson's ratio is
 212 0.3.

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

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

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

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

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

236 *Specimen showing apparent Whitmore fracture*

237 As mentioned in the section "Comparisons between the Whitmore section and the block shear
 238 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 ϵ_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.

296 The Whitmore criterion has been shown in the paper to be excessively conservative for connections
297 having two or three rows of bolts. The ultimate test load obtained by Aalberg & Larsen (1999) for a
298 connection having two rows of bolts was 90% higher than that predicted by the Whitmore criterion.
299 If the Whitmore criterion were valid, such an outcome would not have been possible. The ultimate
300 test loads of the specimens tested by Aalberg & Larsen (1999) were accurately determined using the
301 block shear equation proposed by Teh & Deierlein (2017).

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

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

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

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Figure 1 The Whitmore section: (a) Geometric variables; (b) Whitmore fracture

Figure 2 Difficulty in the Whitmore section concept

Figure 3 Geometric variables of the block shear capacity

Figure 4 True stress-strain curve for Specimen T-16

Figure 5 Calibration of damage initiation parameters for Specimen T-16 (Aalberg & Larsen 1999)

(a)

(b)

Figure 6 Contours of Specimen T-16 at the ultimate limit state: (a) Out-of-plane displacements; (b) Longitudinal normal stresses

Figure 7 von Mises stress contours of Specimen T-16

(a)

(b)

Figure 8 Specimen T-16: (a) Fracture imminent; (b) No Whitmore fracture

Figure 9 Load-deflection graphs of Specimen T-16 (Aalberg & Larsen 1999)

Figure 10 Specimen showing apparent Whitmore tension fracture (Bjorhovde & Chakrabarti 1985)

Figure 11 True stress-strain curve for BJORHOLM & CHAKRABARTI (1985)

Figure 12 Load-deflection graph for Bjorhovde & Chakrabarti (1985)

Figure 13 Necking at the ultimate limit state of Bjorhovde & Chakrabarti (1985)

Figure 14 Complete tensile fracture within the block only

Figure 15 Fracture in the outer Whitmore zone

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

Spec	e_1 (mm)	p (mm)	g (mm)	d_h (mm)	t (mm)	n_r	n_l	F_y (MPa)	F_u (MPa)	P_t/R_n		
										Whitmore Eqn (1)	Block Shear Eqn (2)	FEA
T-1	50	60	65	21	8.4	2	3	373	537	1.58	1.07	1.05
T-3										1.55	1.05	1.03
T-2					7.7			786	822	1.54	1.05	1.02
T-4										1.51	1.02	1.00
T-7	38	47.5	47.5	19	8.4	2	2	373	537	1.90	1.07	1.05
T-8					7.7			786	822	1.79	1.01	0.99
T-9					8.4	3	2	373	537	1.40	1.04	1.02
^a T-15										1.32	0.99	1.03
T-10					7.7			786	822	1.31	0.98	0.96
^a T-16										1.27	0.95	1.00
T-11					8.4	4	2	373	537	1.18	1.00	0.98
T-12					7.7			786	822	1.11	0.94	0.92
								Mean		1.46	1.01	1.00
								COV		0.162	0.043	0.035

^aThese I-section specimens had their flanges removed.

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

[illegible]

Table 3. Ductile damage parameters

Stress Triaxiality	Fracture Strain
0	2
0.20	1.25
0.45	0.5
0.50	0.15
0.57	0.5
0.68	1
1	2

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

Spec	e_1 (mm)	p (mm)	g (mm)	d_h (mm)	t (mm)	n_r	n_l	F_y (MPa)	F_u (MPa)	P_t/R_n	
										Whitmore Eqn (1)	Block Shear Eqn (2)
T1B	38	76	51	21	6.6	3	2	336	450	1.26	1.03
T1C										1.31	1.06
T1A					6.5					1.29	1.05
^b T2B	25						4			1.26	1.12
T2C										1.27	1.12

^bWhile 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)

Spec	e_1 (mm)	p (mm)	g (mm)	d_h (mm)	t (mm)	n_r	n_l	F_y (MPa)	F_u (MPa)	P_t/R_n	
										Whitmore Eqn (1)	Block Shear Eqn (2)
4U	38	76	51	21	6.8	2	2	317	415	1.44	1.07
8U							4			1.13	1.02
12U							6			1.00	0.97
14U							7			0.99	0.99
16U							8			0.93	0.94

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

Spec	P_t/R_n						
	Block Shear Eqn (2)	Whitmore (1951)	Irvan (1957)	Chesson & Munse (1963)	Yamamoto et al. (1985)	Cheng et al. (2000)	Dowswell (2013)
T-1	1.07	1.58	15.8	1.75	1.87	1.15	1.16
T-3	1.05	1.55	15.4	1.71	1.82	1.12	1.13
T-2	1.05	1.54	15.5	1.72	1.83	1.13	1.13
T-4	1.02	1.51	15.1	1.67	1.78	1.10	1.11
T-7	1.07	1.90	4.45	2.27	2.55	1.17	1.18
T-8	1.01	1.79	4.19	2.14	2.40	1.10	1.11
T-9	1.04	1.40	5.84	1.70	1.93	0.83	0.84
T-15	0.99	1.32	5.50	1.60	1.82	0.79	0.79
T-10	0.98	1.31	4.63	1.44	1.65	0.70	0.70
T-16	0.95	1.27	4.37	1.36	1.55	0.66	0.66
T-11	1.00	1.18	5.52	1.60	1.82	0.79	0.79
T-12	0.94	1.11	5.31	1.54	1.76	0.76	0.76