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Ultimate Tilt-Bearing Capacity of Bolted Connections in Cold-Reduced Steel Sheets

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Abstract

This paper examines the accuracy of design equations specified in the North American, European, and Australasian codes for cold-formed steel structures in determining the ultimate tilt-bearing capacity of single-shear bolted connections without washers in flat steel sheets. It points out that all the code equations do not properly distinguish the tilt-bearing failure mode from the conventional bearing failure mode. While the latter takes place downstream of the bolt, the former takes place upstream. Unlike the conventional bearing capacity, the tilt-bearing capacity is affected by the width of the connected sheet and does not vary linearly with either the sheet thickness or the bolt diameter. Furthermore, it is not affected by material ductility. Based on the test results of 156 specimens composed of G2 and G450 sheet steels having various dimensional configurations, this paper proposes a design equation that is dimensionally consistent and that is considerably more accurate than all the code equations. The proposed equation was also verified against single-shear single-row bolted connections tested by independent researchers which failed in the tilt-bearing mode. The verified thicknesses ranged from 0.92 to 3.0 mm, and the bolt diameters ranged from 6.4 to 16 mm. An additional finding is that the tilt-bearing capacity is not significantly affected by the orientation of the bolt head or nut. A resistance factor of 0.75 is recommended for use with the proposed equation for determining the tilt-bearing capacity of single shear single-row bolted connections in cold-reduced steel sheets.

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1 Ultimate Tilt Bearing Capacity of Bolted Connections in Cold-Reduced 2 Steel Sheets

3 Lip H. Teh¹ M.ASCE and Mehmet E. Uz²

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21 with the proposed equation for determining the tilt bearing capacity of single shear single-
22 row bolted connections in cold-reduced steel sheets.

23
24 **Author keywords:** bolted connections, bearing strength, bolt punching, cold-formed steel,
25 connection capacity, connection curling, single-shear bolted connection, tilt bearing

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26 Introduction

27 The ultimate bearing capacity of a bolted connection in cold-formed steel sheet is specified in
28 Section E3.3.1 of the North American Specification for the Design of Cold-formed Steel
29 Structural Members 2012 (AISI 2012), in Table 8.4 of the European code EN-1993-1-3:2006
30 (ECS 2006), and in Clause 5.3.4.2 of the Australasian standard AS/NZS 4600:2005 (SA/SNZ
31 2005). No fundamental distinction is made in any of the design codes between a double-shear
32 and a single-shear connection, although the North American and the Australasian codes
33 employ modification factors that are equal to 1.33 for the inside sheet of a double-shear
34 connection and 0.75 for a single-shear connection without washers. However, the
35 conventional bearing failure mode typical of the inside sheet of a double-shear connection
36 has a fundamentally different mechanism from the tilt bearing failure mode that may be
37 experienced by a single-shear connection without washers, as evident from Figure 1.

38 It can be seen that the conventional bearing failure shown in Figure 1(a) occurred on the
39 downstream side of the bolt hole (i.e. in the direction of bolt displacement), while the tilt
40 bearing failure of the single-shear single-bolt connection without washers in Figure 1(b) was
41 due to the bolt head (and nut) punching through the sheet on the upstream side during tilting
42 under eccentric loading. Figure 2 shows that, in a tilt bearing failure, fracture takes place on
43 the upstream side of the bolt hole, and not on the downstream side.

44 Another distinguishing feature of the tilt bearing failure mode is curling of the single-shear
45 connected sheets, as shown in Figure 3. The curling, which can be observed long before the
46 single-shear connection reaches the ultimate load, and which facilitates bolt tilting, is
47 essentially bending of the sheet under eccentric loading. If curling is absent (or restrained),
48 then it will be more difficult for the bolt head or nut to punch through the connected sheet on
49 the upstream side. This fact indicates that the tilt bearing capacity of a bolted connection does

50 not necessarily vary linearly with the sheet thickness, and that the sheet width is a factor
51 resisting the tilt bearing failure.

52 It should be noted that the present connections, which are without washers, are not
53 comparable to the single-shear connections tested by Bryan (1993). The presence of washers
54 in the specimens of Bryan (1993) ensured that failures occurred on the downstream side of
55 the bolt hole, which could even be in the form of shear-out (or tear-out). Likewise, the
56 bearing failures of the single-shear connections tested by Yan & Young (2011) took place on
57 the downstream side due to the use of washers and clips that prevented independent curling
58 of the connected sheets. The finite element models of Yan & Young (2012), on the other
59 hand, were prevented from out-of-plane displacements in the connection region.

60 When no washer nor clip is used for a single-shear bolted connection in thin steel sheet, there
61 is another failure mode that was associated by some researchers with tilting and bearing. The
62 failure mode is depicted in Figure 4, and was experienced by many of the specimens tested
63 by Carril et al. (1994) and Casafont et al. (2006). Such a failure mode was called “localised
64 tearing” by Rogers & Hancock (2000), who used clips and observed that this failure mode
65 only took place in the more ductile mild sheet steels. It should be noted that the localised
66 tearing mode may take place with or without washers/clips. This failure mode may also
67 happen to a serial bolted connection, as shown by Rogers & Hancock (2000), and was in the
68 past mistaken to be the net section tension fracture mode as discussed by LaBoube (1988)
69 and Rogers & Hancock (2000). The localised tearing mode is outside the scope of this paper.

70 The authors did not detect evidence of tilt bearing failures as studied in this paper among the
71 specimens tested by Wallace & Schuster (2001). Figure 6(a) of Wallace & Schuster (2001)
72 shows a bearing failure on the downstream side of the bolt hole of a specimen without
73 washers despite the presence of curling, as also described in their report. This failure mode is

74 similar to that shown in the top right photo of Fig. 8 of Talja & Torkar (2014), which shows
75 the failure of a single-shear screwed connection with an integrated washer. Yu & Mosby
76 (1981), who tested single-shear bolted connections in thin sheets, did not discuss the tilt
77 bearing failure mode shown in Figures 1(b) and 2.

78 Rogers & Hancock (2000) did not describe the failure mode that is due to the bolt head and
79 nut punching through the connected sheets on the upstream side of the respective bolt holes
80 during tilting. However, they mentioned the “bearing with end curling” mode, which was
81 associated with fracture on the downstream side of the bolt hole. Unlike the tilt bearing
82 failure mode studied in this paper, the bearing with end curling mode can happen to bolted
83 connections with washers, and without bolt tilting as depicted in Figure 5.

84 The tilt bearing failure mode has not been described in any design code or guidelines either.
85 The North American and the European guidelines on the testing of sheet steel connections
86 (AISI 2008, ECCS 2009) describe five failure modes including the so-called “tilting and pull-
87 out failure” mode, but do not mention the tilt bearing failure shown in Figures 1(b) and 2.
88 Likewise, Yu & Panyanouvong (2013) depicted four failure modes for single-shear bolted
89 connections, but did not include the tilt bearing failure mode studied in this paper.

90 This paper presents the first ever systematic study on the tilt bearing capacities of bolted
91 connections, which are due to the bolt head and nut punching through the connected sheets on
92 the upstream side of the respective bolt holes during tilting, an example of which is shown in
93 Figure 1(b). The failure mode has been observed for single-shear single-row bolted
94 connections without washers in cold-reduced steel sheets only, which are often used in
95 bracing applications of light gauge cold-formed steel frames. The paper details how a
96 nonlinear empirical equation for the ultimate tilt bearing capacity can be derived
97 methodically without losing dimensional consistency. The design equation will be formulated

98 based on the results of 156 G2 and G450 sheet steel specimens having various dimensional
99 configurations tested in the present work, and verified against independent test results of
100 other researchers (Casafont et al. 2006, Yu & Sheerah 2008, Hoang et al. 2013) where the
101 single-shear single-row bolted connection specimens are known to have failed by tilt bearing.

102 Interested readers may consult Teh & Uz (2014) for the conventional bearing failure mode,
103 Teh & Gilbert (2012) for the net section tension fracture mode, Teh & Clements (2012) for
104 the block shear failure mode, and Teh & Uz (2015) for the shear-out failure mode. The
105 definitions may differ from those used by some other researchers, and the authors believe that
106 the definitions by Teh and co-workers are correct and consistent.

107 Bolt hole deformation at service load is not a concern in this paper. The tilt bearing capacity
108 of a specimen is defined as the maximum test load achievable.

109 **Equations for bearing capacity of single-shear bolted connection**

110 Section E3.3.1 of the North American Specification for the Design of Cold-formed Steel
111 Structural Members 2012 (AISI 2012) specifies the bearing capacity per bolt of a single-shear
112 bolted connection without washers to be

$$113 \quad P_b = 0.75C d t F_u \quad (1)$$

114 in which d is the bolt diameter, t is the sheet thickness and F_u is the material tensile strength.
115 The bearing factor C depends on the ratio of the bolt diameter d to the sheet thickness t , as
116 shown in Table 1. It may be noted that a ratio d/t greater than 10 is rare in practice.

117 According to the North American specification (AISI 2012), the “modification factor” of 0.75
118 in Equation (1) differentiates a single-shear connection from a double-shear one (1.33), and
119 accounts for the absence of washers (1.00). These values were obtained by Wallace &

120 Schuster (2002) through statistical analysis of the test results of Wallace & Schuster (2001).
121 All other variables in Equation (1) are the same between the various connection
122 configurations despite the very different mechanisms evident in Figure 1 between the tilt
123 bearing failure possible for single-shear single-row bolted connections without washers and
124 the conventional bearing failure typical of double-shear connections or single-shear
125 connections with washers. However, it is most unlikely that the different failure mechanisms
126 can be adequately accounted for using only the modification factors defined in Section E3.3.1
127 of the specification (AISI 2012).

128 It can be seen from the statistical results of Wallace & Schuster (2002) that Equation (1) leads
129 to a large coefficient of variation equal to 0.151, meaning that on average the error is about
130 15% on either side of conservatism. One reason is already mentioned in the preceding
131 paragraph. Another possible reason is that some of the analysed specimens failed in the tilt
132 bearing mode shown in Figure 1(b) while others failed in bearing on the downstream side of
133 the bolt hole, as shown in Figure 6(a) of Wallace & Schuster (2001).

134 In any case, the Australasian cold-formed steel structures standard (SA/SNZ 2005) adopts
135 Equation (1) for a single-shear bolted connection without washers. On the other hand, the
136 European code EN-1993-1-3:2006 (ECS 2006) does not make a distinction between single
137 and double shear connections, and does not consider the benefit of washers. For the
138 specimens tested in the present work, in which the end distance was invariably more than 3
139 times the bolt diameter, the European code specifies the bearing strength per bolt to be

$$140 \quad P_b = 2.5k_t d t F_u \quad (2)$$

141 in which the variable k_t is equal to unity for sheet thicknesses greater than 1.25 mm,
142 otherwise it is

$$143 \quad k_t = \frac{0.8t + 1.5}{2.5}; \quad 0.75 \text{ mm} \leq t \leq 1.25 \text{ mm} \quad (3)$$

144 For Equation (3) to be valid (not necessarily accurate), not only the metric system must be
 145 used, but the sheet thickness t must also be measured in millimetres since the two constants
 146 are dimensionless. Such a dimensionally inconsistent equation should be avoided as much as
 147 possible since it is prone to errors in design calculations.

148 Equations (1) and (2) imply that the “bearing capacity” of a single-shear single-row bolted
 149 connection varies linearly with the sheet thickness and the bolt diameter, in the same manner
 150 as that of a double-shear connection. However, such simple relationships are unlikely to hold
 151 when the failure is due to the bolt head and nut punching through the connected sheets on the
 152 upstream side of the respective bolt holes during tilting. Furthermore, the sheet width W is
 153 likely to influence the tilt bearing capacity as the resistance to curling and therefore bolt
 154 tilting increases with increasing width, yet this parameter is absent in both code equations.

155 In the present work, the tilt bearing capacity per bolt of a single-shear single-row bolted
 156 connection is expressed as

$$157 \quad P_b = C_{tb} d^b t^a W_n^c F_u \quad (4)$$

158 in which W_n is the sheet width that is net of the bolt hole diameter. The geometric variables
 159 are defined in Figure 6. For single-row bolted connections having more than one bolt, the net
 160 sheet width W_n is equal to the total net sheet width divided by the number of bolts.

161 The ultimate tilt bearing coefficient C_{tb} and the exponential terms a through c would be
 162 determined through analyses of the present test results, and verified against independent test
 163 results of other researchers whose specimens are known to have failed in tilt bearing due to

164 the bolt head punching through the connected sheet on the upstream side of the bolt hole
165 (Casafont et al. 2006, Yu & Sheerah 2008, Hoang et al. 2013).

166 In order to ensure dimensional consistency, the sum of the exponential terms a , b and c must
167 be equal to 2. Since the least dominant geometric variable on the tilt bearing capacity is the
168 net sheet width W_n , which is absent in the code equations, the exponential term c is
169 determined solely as a function of a and b

$$170 \quad c = 2 - (a + b) \quad (5)$$

171 **Test materials**

172 The G450 and G2 sheet steel materials used in the present laboratory tests, which have trade
173 names GALVSPAN[®] and GALVABOND[®], respectively, were manufactured and supplied
174 by Bluescope Steel Port Kembla Steelworks, Australia. G2 sheet steel is classified as a
175 formability grade, while G450 sheet steel is a structural grade (SA 2011).

176 The average yield stresses F_y , tensile strengths F_u and elongations at fracture over 15 mm, 25
177 mm and 50 mm gauge lengths ϵ_{15} , ϵ_{25} and ϵ_{50} , and uniform elongation outside the fracture ϵ_{u0}
178 of the steel materials as obtained from 12.5 mm wide tension coupons are shown in Tables 2
179 and 3 for the G450 and G2 sheet steels, respectively. The variable t_{base} denotes the base metal
180 thickness without the coating. Tensile loadings of all coupons and bolted connection
181 specimens are in the rolling direction of the sheet steel, as required for structural grade sheet
182 steels (SA 2011). The tension coupon tests were conducted at a constant stroke rate of 1
183 mm/minute resulting in a strain rate of about 2×10^{-4} per second prior to necking.

184 Tables 2 and 3 show that the G2 sheet steel is considerably more ductile than the G450 sheet
185 steel. The 1.9-mm and 2.4-mm G450 sheet steels just meet the requirements for being used
186 without restriction according to Section A2.3.1 of the North American specification (AISI

187 2012), while the 1.5-mm and 3.0-mm ones marginally fail them. However, G450 sheet steel
188 is a structural grade covered by the Australasian standard (SA/SNZ 2005) for which the
189 nominal tensile strength and yield stress may be fully utilised in structural design calculations
190 (Hancock 2007).

191 **Specimen configurations and test arrangements**

192 Almost all specimens tested in the present work were single-shear single bolted connections,
193 as depicted in Figure 3. The distance between each bolt and the downstream end was at least
194 50 mm to prevent the shear-out failure mode. Each specimen was only lightly tightened by
195 hand.

196 For the purpose of determining the relationship between the sheet thickness and the tilt
197 bearing capacity, the present work tested fifty seven G450 sheet steel specimens having
198 nominal thicknesses of 1.5, 1.9, 2.4 and 3.0 mm. The resulting equation would be verified
199 against the test results of Yu & Sheerah (2008) involving 0.92 mm Grade 33 and 1.12 mm
200 Grade 50 sheet steels.

201 In order to ascertain the effect of sheet width, for almost every thickness of the G450 sheet
202 steels, the widths were 50, 60, 70, 75, 100 and 120 mm. These values represent the range that
203 may be covered by one bolt in cold-formed steel constructions. The 120-mm wide specimens
204 with 12-mm bolt represent a bolt spacing of 10 times the bolt diameter, which is unlikely to
205 be exceeded in practice. The derived equation would be verified against the test results of Yu
206 & Sheerah (2008) involving a ratio of sheet width to bolt diameter W/d close to 16.

207 Two bolt sizes commonly used for structural connections in G450 sheet steels, 12 and 16
208 mm, were used in the present tests, resulting in the ratio of bolt diameter to sheet thickness d/t
209 ranging from 4.0 to 10.7. The proposed equation would be verified against the test results of

210 Yu & Sheerah (2008) involving 6.4 mm bolts, and those of Casafont et al. (2006) and Hoang
211 et al. (2013) involving 8 mm bolts.

212 According to Section E3a of the North American specification (AISI 2012), the standard hole
213 diameter shall not be more than 1 mm greater than the bolt diameter for bolt sizes up to 13
214 mm, or more than 2 mm greater than the bolt diameter for bolt sizes over 13 mm. Similar
215 clearances are specified in Clause 5.3.1 of the Australasian standard (SA/SNZ 2005), except
216 that the threshold bolt size is 12 mm. The specimens of Yu & Sheerah (2008) which had
217 significantly larger hole clearances, i.e. oversized holes, are not included in the present
218 verification unless commensurately oversized bolt heads were used.

219 The fifty seven specimens whose results would be used to determine the relationships
220 between the tilt bearing capacity and each of the three geometric variables had a nominal bolt
221 hole clearance of 2 mm, the absolute maximum allowed by the codes (AISI 2012, SA/SNZ
222 2005). The effect of smaller bolt hole clearance would be investigated by testing twenty nine
223 G450 and thirty two G2 specimens having 1 mm clearance only.

224 A total of sixty four specimens composed of G2 sheet steel, which has very different ductility
225 characteristics from G450 sheet steel as evident from Tables 2 and 3, would be tested in light
226 of the finding of Teh & Uz (2014) regarding the effect of material ductility on the bearing
227 capacity of double-shear bolted connections. Importantly, the G2 specimens also provided an
228 opportunity to investigate the effect of the orientation of bolt head and nut on the tilt bearing
229 capacity. The two orientations are shown schematically in Figure 7, with actual samples
230 shown in Figure 8.

231 The measured dimensions of the 12-mm and 16-mm bolt heads are shown in Figure 9. These
232 parameters are important since it is the bolt head and nut that punch through the sheet.

233 Supplementing the one hundred fifty single-bolt connection specimens tested in the present
234 work, a total of six single-shear single-row double bolted connection specimens were also
235 tested. An example is shown in Figure 10.

236 For all specimens, the two connected sheets were nominally identical to each other. This
237 arrangement means that the equation derived in the present work for determining the ultimate
238 tilt bearing capacity represents the lower bound when the equation is applied to the weakest
239 sheet.

240 **Mechanism of a tilt bearing failure**

241 The properties of the specimens cited in the following paragraphs are given in Tables 4
242 through 8 discussed in the next section, but they are not essential to the discussions in this
243 section. The label designations of the specimens mainly followed the order of testing.

244 Figure 11 shows a bolt head punching through the bolt hole of Specimen ES34, which
245 fractured on the upstream side as can be seen in the photograph when examined closely. The
246 load-displacement graph of the specimen is plotted in Figure 12, which shows a little kink
247 associated with the fracture before the applied load gradually dropped. It is therefore
248 tempting to conclude that the ultimate tilt bearing capacity of a bolted connection is reached
249 whenever fracture takes place on the upstream side of either bolt hole, in a similar manner
250 that the net section tension capacity is associated with fracture of the weakest sheet.
251 However, Figure 12 does not show the typical load-displacement behaviour of the specimens
252 tested in the present work.

253 Figure 13 demonstrates that Specimen ES37a, shown in Figure 1(b), was able to recover after
254 the initial drop in the applied load following fracture on the upstream side of the bolt hole as
255 the bolt head punched through. The specimen reached a second peak load before its load-

256 carrying capacity definitely decreased. In fact, such a behaviour is not uncommon among the
257 tested specimens as shown in Figure 13 for Specimen ES37b and in Figure 14 for Specimens
258 ES46a, ES49b and YK21. It appears that, for these specimens, the ultimate load capacity was
259 only reached after fractures took place on the upstream side of both bolt holes (both sheets).

260 Unlike other failure modes such as the net section tension fracture mode and the block shear
261 failure mode, a fracture on the upstream side of only one bolt hole (one sheet) does not
262 necessarily mean that the connection has reached its maximum resistance to bolt tilting under
263 the eccentric loading (see Figure 3 for illustration). The other, still intact sheet is capable of
264 providing further resistance to bolt tilting, so the connection is able to carry increased loading
265 until fracture takes place in the second sheet (it should be noted that there is no fracture on
266 the downstream side of either bolt hole).

267 In any case, it is evident that the sheet width must affect the ultimate tilt bearing capacity of a
268 single-shear single-row bolted connection since the wider the sheet, the greater its available
269 resistance to curling and therefore to tilting. This assertion is supported by the test results
270 presented in Tables 4 through 9 discussed in the next section.

271 It is noteworthy that some load-displacement graphs do not exhibit a noticeable kink, as
272 shown in Figure 13 for Specimen ES37b, and in Figure 15 for Specimens ES12 and YK1, the
273 latter two only experiencing one peak load each like Specimen ES34 discussed previously.
274 Another interesting thing is that the three specimens plotted in Figure 14 did not have
275 significantly different initial response despite their different geometry, in contrast to the
276 comparison between Specimens ES12 and YK1 in Figure 15. Advanced finite element
277 analysis is required to investigate the issues thoroughly.

278 **Exponential terms a , b and c**

279 Tables 4 and 5 lists the geometric dimensions and ultimate test loads of G450 specimens that
 280 had a nominal bolt hole clearance of 2 mm, for 12-mm and 16-mm bolts, respectively. For
 281 legibility, only the nominal values of the geometric variables are shown in the tables.
 282 However, for calculation purposes, the measured values and the base metal thicknesses listed
 283 in Tables 2 and 3 were used. An empty cell in a table indicates that the data in the above cell
 284 applies.

285 The variable P_t in the tables denotes the ultimate test load obtained in the experimental
 286 program, while P_p is the tilt bearing capacity predicted by the equations. The ratio P_t/P_p is
 287 called the professional factor. It can be seen that Equations (1) and (2), which are premised
 288 on the conventional bearing failure on the downstream side of the bolt, considerably
 289 overestimated the ultimate tilt bearing capacity. Everything else remaining the same, the
 290 overestimations decrease with increasing sheet thicknesses. This observation indicates that
 291 the tilt bearing capacity increases exponentially with the sheet thickness.

292 The variations of the tilt bearing capacity with the sheet thickness were checked against 12
 293 groups of specimens across the nominal thicknesses ranging from 1.5 mm to 3.0 mm in each
 294 group, as shown in Tables 4 and 5. The only (nominal) geometric variable in each group is
 295 the sheet thickness t , with the bolt diameter d and the sheet width W remaining constant. In
 296 most groups, the reference (nominal) thickness is 1.5 mm. The normalised capacity ratio r_{th}
 297 shown in the tables were calculated from

$$298 \quad r_{th} = \frac{P_t F_{u\text{ref}} t_{\text{ref}}}{P_{\text{ref}} F_u t} \quad (6)$$

299 An entry “Ref” in Tables 4 or 5 indicates that the specimen is the reference specimen for the
 300 group of specimens having the same bolt diameter and the same nominal width W , whose
 301 properties and ultimate test load are identified by the subscript “ref” in Equation (6).

302 As can be seen in Tables 4 and 5, some configurations were tested twice, and the specimens
 303 were denoted by the prefix “a” or “b” in their respective labels. For such specimens, only
 304 their average values were used in the calculations.

305 Figure 16 plots the normalised capacity ratios r_{th} for three groups of specimens found in
 306 Tables 4 ($d = 12$ mm) and 5 ($d = 16$ mm) that covered the four nominal thicknesses. If the
 307 actual tilt bearing capacity varied linearly with the sheet thickness, then the ratios r_{th} would
 308 be constant at unity across the different thicknesses. However, as shown in the figure, the
 309 normalised capacity ratios increased with the sheet thicknesses, which means that the tilt
 310 bearing capacity increases more rapidly than the sheet thickness.

311 In any case, the exponential term a in Equation (4) should satisfy

$$312 \quad r_{th} = \left(t / t_{ref} \right)^{a-1} \quad (7)$$

313 In order to avoid a decimal exponential term in Equation (4) if feasible, the exponential term
 314 a is taken to have the following form

$$315 \quad a = 1 + \frac{i}{j} \quad (8)$$

316 in which i and j are positive integers. The measured base thicknesses listed in Tables 2 and 3
 317 were used in determining the exponential term a .

318 Based on the test results shown in Tables 4 and 5, it was found that using $a = 4/3$ simulated
 319 the relationship between the tilt bearing capacity and the sheet thickness quite well. It can be
 320 shown that the resulting mean “professional factor” for the ratios r_{th} is 1.02 with a coefficient
 321 of variation equal to 0.062.

322 The relationship between the tilt bearing capacity and the bolt diameter was investigated next.
323 Strictly speaking, the relevant parameter is the bolt head size rather than the bolt shank
324 diameter since it is the bolt head (and nut) that punches through the connected sheet.
325 However, as can be computed using the data given in Figure 9, for the 12-mm and 16-mm
326 bolts used in the present work, the ratio of the bolt shank diameters is the same as that of the
327 bolt head sizes, so it does not matter which parameter is used for determining the relationship
328 between the tilt bearing capacity and the bolt diameter. However, in practice it is common to
329 use the bolt shank (nominal) diameter rather than the bolt head size in designing a bolted
330 connection.

331 The variable r_d in Table 5 denotes the ratio between the ultimate test load of a 16-mm bolt
332 specimen and that of the corresponding 12-mm bolt specimen, the latter given in Table 4. The
333 average test value of the ratio r_d is 1.16, with a coefficient of variation equal to 0.065. It was
334 found that the use of an exponential term b in Equation (4) equal to $\frac{1}{2}$ would give a ratio of
335 1.15. The exponential term b is therefore taken to be $\frac{1}{2}$, meaning that the tilt bearing capacity
336 varies with the square root of the bolt diameter.

337 Incidentally, the use of $b = \frac{1}{2}$ is consistent with a limiting equation for the “tilting and hole
338 bearing” capacity of a screwed connection specified in Clause 5.4.2.3 of the Australasian
339 cold-formed steel structures standard (SA/SNZ 2005). This clause corresponds to Section
340 E4.3.1 of the North American specification (AISI 2012), which uses the term “shear strength
341 limited by tilting and bearing”. It may be noted that the limiting equation has the strength of a
342 screwed connection varying with the sheet thickness at the power of $\frac{3}{2}$. The sheet width is
343 not a parameter in the specification equation.

344 Having determined the exponential terms a and b to be $\frac{4}{3}$ and $\frac{1}{2}$, respectively, the
345 exponential term c was computed from Equation (5) to be $\frac{1}{6}$.

346 **Ultimate tilt bearing coefficient**

347 Table 6 lists the geometric dimensions and ultimate test loads of the G450 specimens which
348 had a nominal bolt hole clearance of 1 mm. It was found that the tighter hole clearance
349 increased the ultimate tilt bearing capacity by about 5% only on average, justifying the use of
350 one tilt bearing coefficient C_{tb} common to all bolt holes having clearances up to the
351 maximum of 2 mm allowed by the codes (AISI 2012, SA/SNZ 2005).

352 Tables 7 and 8 list the geometric dimensions and ultimate test loads of G2 specimens that had
353 nominal bolt hole clearances of 2 mm and 1 mm, respectively. By comparing the professional
354 factors in these tables against those in Tables 4 through 6, it can be concluded that the
355 significantly different levels of material ductility between G2 and G450 sheet steels, as
356 evident from Tables 2 and 3, did not affect the tilt bearing capacities. This conclusion
357 supports the assertion that the tilt bearing failure mode shown in Figure 1(b) has a
358 fundamentally distinct mechanism from the conventional bearing failure mode shown in
359 Figure 1(a), since the latter is significantly affected by material ductility (Teh & Uz 2014).

360 The results shown in Tables 7 and 8 also indicate that the orientations of the bolt head and nut
361 did not have significant effect on the ultimate tilt bearing capacity, although there was some
362 5% difference on average for the specimens having a nominal hole clearance of 2 mm. For
363 the specimens having a nominal hole clearance of 1 mm, there were no consistent differences
364 in the tilt bearing capacities. This finding avoids a potential complication for the design
365 equation. The orientations I and II specified in the tables are shown in Figures 7 and 8. It may
366 be noted that the specimens listed in Tables 4 through 6 had random orientations.

367 Having established that variations in bolt hole clearances (within code limits), material
368 ductility, and bolt head/nut orientation do not have meaningful effects on the ultimate tilt

369 bearing capacity of single-shear single-row bolted connections, the ultimate tilt bearing
370 coefficient C_{tb} in Equation (4) was determined to be 2.65 (rounded to the nearest 0.05) based
371 on the ultimate test loads of 150 specimens listed in Tables 4 through 8 and the exponential
372 terms a , b and c computed in the preceding section. Equation (4) therefore becomes

$$373 \quad P_b = 2.65 d^{1/2} t^{4/3} W_n^{1/6} F_u \quad (9)$$

374 **Verifications of the proposed equation**

375 The professional factors of Equation (9) for the present test specimens are given in Tables 4
376 through 9. Table 9 contains the results of single-shear single-row double bolted connection
377 specimens tested to verify the use of Equation (9) where the variable W_n is equal to the total
378 net sheet width divided by two.

379 Equation (9) was also checked against the test results of independent researchers where the
380 specimens failed by tilt bearing due to the bolt head punching through the connected sheet on
381 the upstream side of the bolt hole, and where the nominal hole diameter clearance did not
382 exceed 2 mm. Yu & Sheerah (2008) tested 12 such specimens composed of Grade 33 and
383 Grade 50 sheet steels with a diameter clearance of 1.6 mm on 1/4-inch A307 bolt, the results of
384 which are shown in the first twelve rows of Table 10.

385 Table 10 shows that Equations (1) and (9) give similar professional factors for the specimens
386 tested by Yu & Sheerah (2008). In any case, it can be seen that Equation (9) is reasonably
387 accurate if somewhat conservative for the 0.92 mm thick specimens, each of which was
388 connected by a 6.4 mm bolt. It is quite accurate for the 1.12 mm thick specimens.

389 Casafont et al. (2006) tested single-shear single-row bolted connections having two bolts
390 each, similar to the specimen shown in Figure 10. From the photographs provided in their

391 paper, most of the specimens appear to have failed in the localised tearing mode depicted in
392 Figure 4. However, one specimen, shown in Figs. 31 and 32 of their paper, failed in tilt
393 bearing due to the bolt head punching through the connected sheet on the upstream side of the
394 bolt hole. The S250 steel specimen, which had a bolt hole clearance of 1 mm on 8-mm bolt,
395 is included in Table 10. Equation (9) was found to be about 10% conservative for this
396 specimen.

397 Hoang et al. (2013) tested only one specimen, composed of 6082 T6 aluminium. Table 10
398 shows that Equation (9) was found to be accurate for this aluminium specimen, which had a
399 bolt hole clearance of 0.5 mm on 8-mm bolt.

400 **Oversized hole and oversized bolt head**

401 As indicated by the report's title, Yu & Sheerah (2008) tested bolted connections with
402 oversized and slotted holes. Equation (9) may not be applicable to connections with a hole
403 diameter clearance significantly greater than 2 mm unless the bolt head size is
404 commensurately larger.

405 For their specimens having a bolt hole clearance of 3.2 mm on ½-inch (12.7 mm) bolt, two
406 types of bolts were used, being A307 and A325 bolts. The A307 bolt had a measured head
407 width s of 18.8 mm, comparable to that of the 12-mm bolt used in the present tests
408 considering the bolt diameters. Equation (9) is therefore not applicable to the specimens
409 having a bolt hole clearance of 3.2 mm on A307 bolt. However, the A325 bolt had a
410 measured head width s of 21.9 mm, which more than offset the enlarged hole clearance.

411 Table 11 lists the geometric dimensions and results of the specimens tested by Yu & Sheerah
412 (2008) which had a hole diameter clearance of 3.2 mm on A325 bolt and which failed in tilt
413 bearing due to the bolt head punching through the connected sheet on the upstream side of the

414 bolt hole. It can be seen that, in contrast to the code equations, Equation (9) is reasonably
 415 accurate for these specimens. In this case, the oversized bolt head offset the effect of the
 416 oversized bolt hole.

417 **Resistance factor (or capacity reduction factor)**

418 The mean professional factors P_m given by Equations (1), (2) and (9) for the 170 specimens
 419 listed in Tables 4 through 10 are given in Table 12, along with their coefficients of variation.
 420 The specimens with 3.2 mm hole clearance on A325 bolt, listed in Table 11, are not included
 421 in the present reliability analysis that is restricted to bolt holes with a maximum clearance of
 422 2 mm.

423 Section F1.1 of the North American specification (AISI 2012) specifies that the resistance
 424 factor ϕ of a design equation is determined as follows

$$425 \quad \phi = C_\phi (M_m F_m P_m) e^p \quad (10)$$

426 in which C_ϕ is the calibration coefficient equal to 1.52 in the case of the Load and Resistance
 427 Factor Design (LRFD), M_m is the mean value of the material factor equal to 1.10 according to
 428 Table F1 of the North American specification, F_m is the mean value of the fabrication factor
 429 equal to 1.00, and P_m is the mean value of the professional factor given in Table 12 for the
 430 concerned design equation.

431 The power p of the natural logarithmic base e in Equation (10) is

$$432 \quad p = -\beta_0 \sqrt{V_M^2 + V_F^2 + C_p V_P^2 + V_Q^2} \quad (11)$$

433 in which V_M is the coefficient of variation of the material factor equal to 0.08 in the present
 434 case, V_F is the coefficient of variation of the fabrication factor equal to 0.05, V_P is the

435 coefficient of variation of the professional factor given in Table 12 for the concerned design
436 equation, C_p is the correction factor equal to 1.01 as computed from the relevant equation
437 given in Section F1.1, and V_Q is the coefficient of variation of load effects equal to 0.21 as
438 specified in Section F1.1.

439 It was found that in order to achieve the target reliability index β_0 of 3.5 in the LRFD,
440 Equation (10) yields a resistance factor of 0.73 for the proposed Equation (9). Given the wide
441 ranges of geometric and material variables included in the analysis, a resistance factor ϕ equal
442 to 0.75 (rounded to the nearest 0.05) in conjunction with Equation (9) is therefore
443 recommended for the LRFD approach for determining the ultimate tilt bearing capacity of a
444 single-shear single-row bolted connection in flat steel sheets.

445 **Conclusions**

446 This paper has pointed out that the tilt bearing failure mode that may be experienced by a
447 single-shear single-row bolted connection without washers is very distinct from the
448 conventional bearing failure mode typical of a double-shear connection or a single-shear
449 connection with washers. The tilt bearing failure is due to the bolt head and nut punching
450 through the connected sheets on the upstream side of the respective bolt holes during tilting,
451 while the conventional bearing failure takes place on the downstream side.

452 Unlike the conventional bearing capacity, it has been found that the tilt bearing capacity is not
453 affected by the variation in material ductility of cold-reduced sheet steels. Furthermore,
454 unlike other failure modes such as the net section tension fracture mode, fracture on the
455 upstream side of one bolt hole only does not necessarily coincide with the maximum load-
456 carrying capacity of the single-shear bolted connection without washers.

457 The paper has also pointed out that the tilt bearing failure mode is distinct from the localised
458 tearing mode, which may also be experienced by a single-shear bolted connection with or
459 without washers, and which was mistaken to be the net section tension fracture mode in the
460 past. The localised tearing mode appears to be more familiar to researchers in cold-formed
461 steel bolted connections.

462 This paper has presented the first systematic study on the tilt bearing capacities of single-
463 shear single-row bolted connections, which were due to the bolt head and nut punching
464 through the connected sheets on the upstream side of the respective bolt holes during tilting.
465 It has been found that the tilt bearing capacity varies nonlinearly with the sheet thickness with
466 an exponent equal to $4/3$, and is proportional to the square root of the bolt diameter. The
467 proposed design equation includes the sheet width as a parameter of the tilt bearing capacity.

468 The proposed design equation is dimensionally consistent and is independent of the systems
469 of measurement or dimensional units that may be used by the structural engineer. It has been
470 found that the equation is reasonably accurate for 170 specimens tested by the authors and
471 other researchers around the world, comprising specimens having sheet thicknesses ranging
472 from 0.92 mm to 3.0 mm and bolt diameters ranging from 6.4 mm to 16 mm with hole
473 clearances ranging from 0.5 mm to 2.0 mm. The tested ratios of sheet width to bolt diameter
474 W/d ranged from 3 to 16. The accuracy of the proposed design equation has not been found to
475 be significantly affected by the orientations of the bolt head and nut.

476 The equation has also been found to be reasonably accurate for specimens each having an
477 oversized hole clearance of 3.2 mm when the A325 bolt type, which has an oversized head
478 compared to the A307 bolt type, was used.

479 It is recommended that a resistance factor of 0.75 be applied to the proposed design equation
480 in the LRFD approach of the North American specification for the design of cold-formed
481 steel structures.

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489 Mulholland-Brown, two honours thesis students at the University of Wollongong.

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(a)



(b)

Figure 1 Distinct failure modes: (a) Bearing failure of a double-shear connection; (b) Tilt bearing failure of a single-shear connection

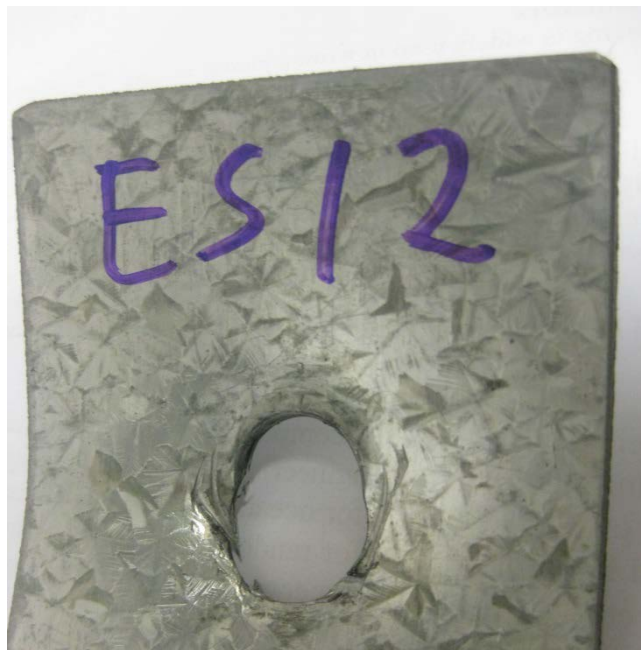


Figure 2 Fracture on the upstream side of bolt hole

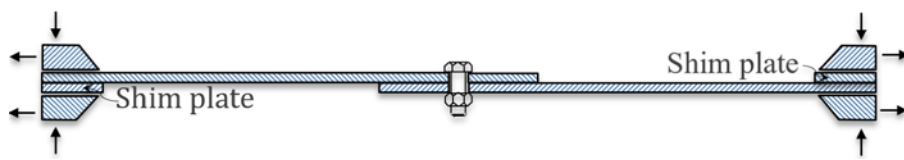


Figure 3 Tilting of a single-shear connection due to load eccentricity



Figure 4 Localised tearing of a single-shear bolted connection (adapted from Casafont et al. 2006)

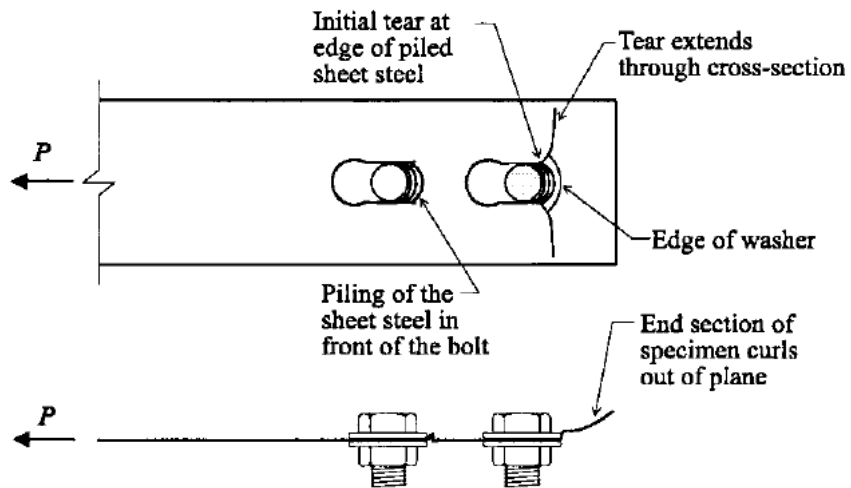


Figure 5 "Bearing with end curling" (Rogers & Hancock 2000)

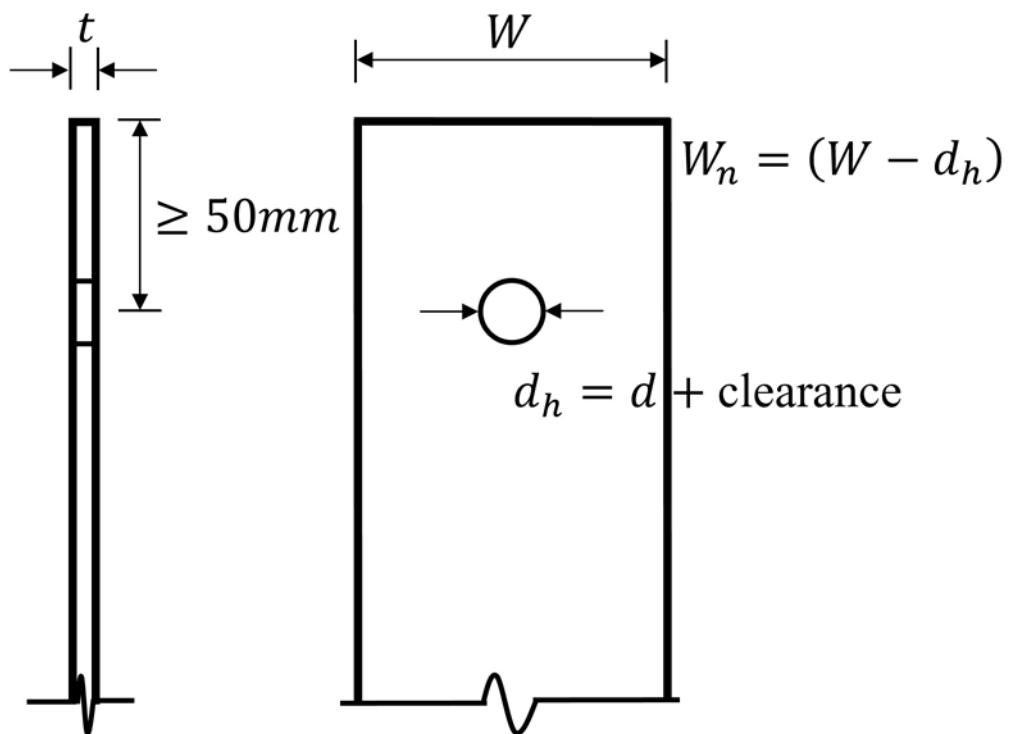


Figure 6 Geometric variables of single-bolt specimens

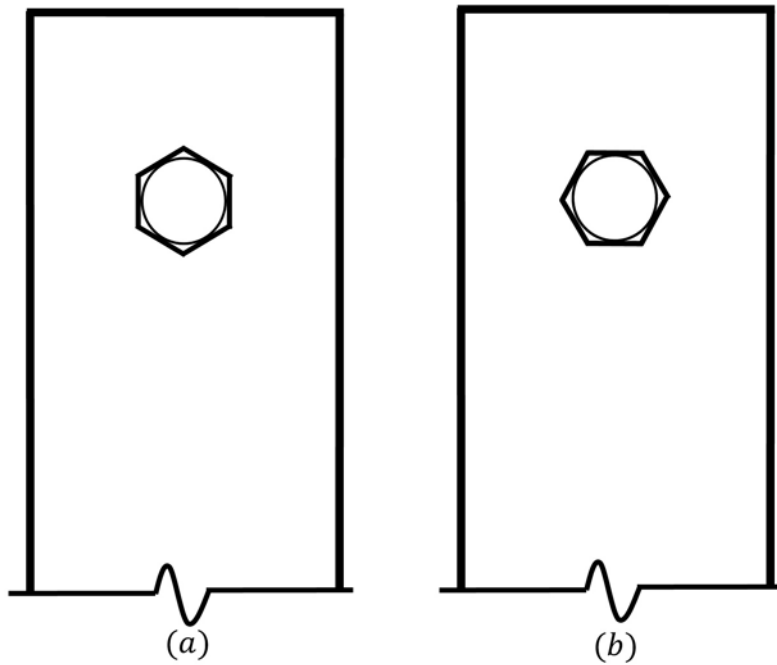


Figure 7 Two different orientations of bolt head and nut: (a) Orientation I; (b) Orientation II



(a)



(b)

Figure 8 Specimens with bolt head (and nut) having (a) Orientation I; (b) Orientation II

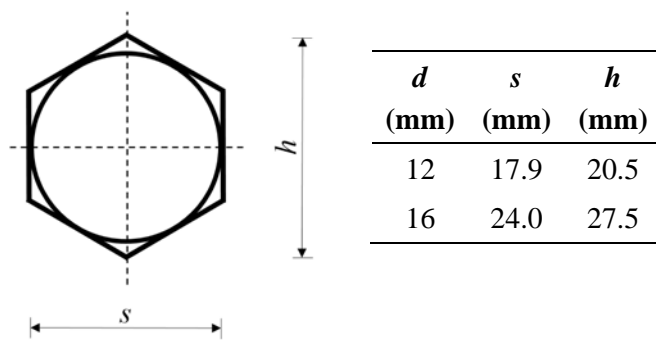


Figure 9 Measured dimensions of tested bolt heads

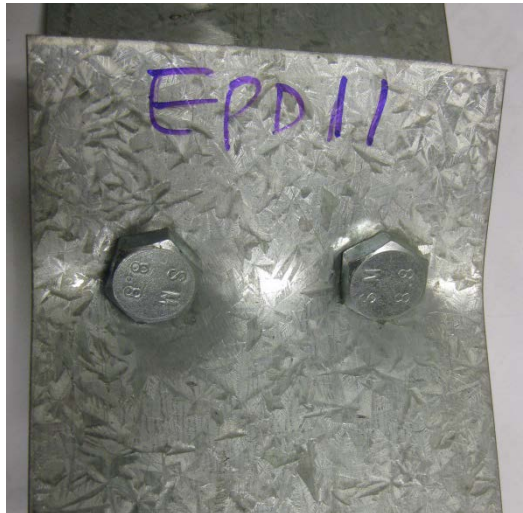


Figure 10 Single-shear single-row double bolted connection specimen



Figure 11 Bolt head punching through the bolt hole of Specimen ES34

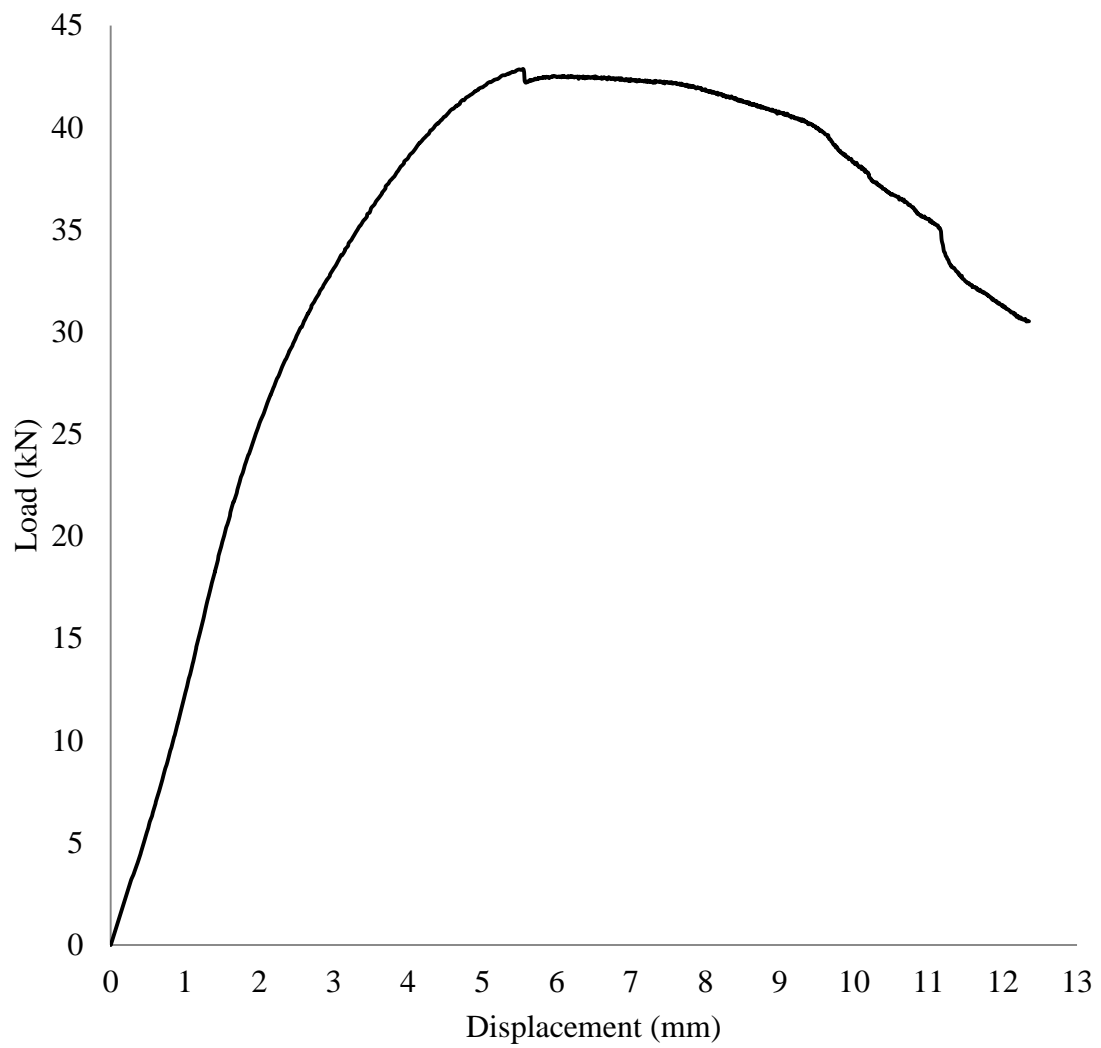


Figure 12 Load-displacement graph of Specimen ES34

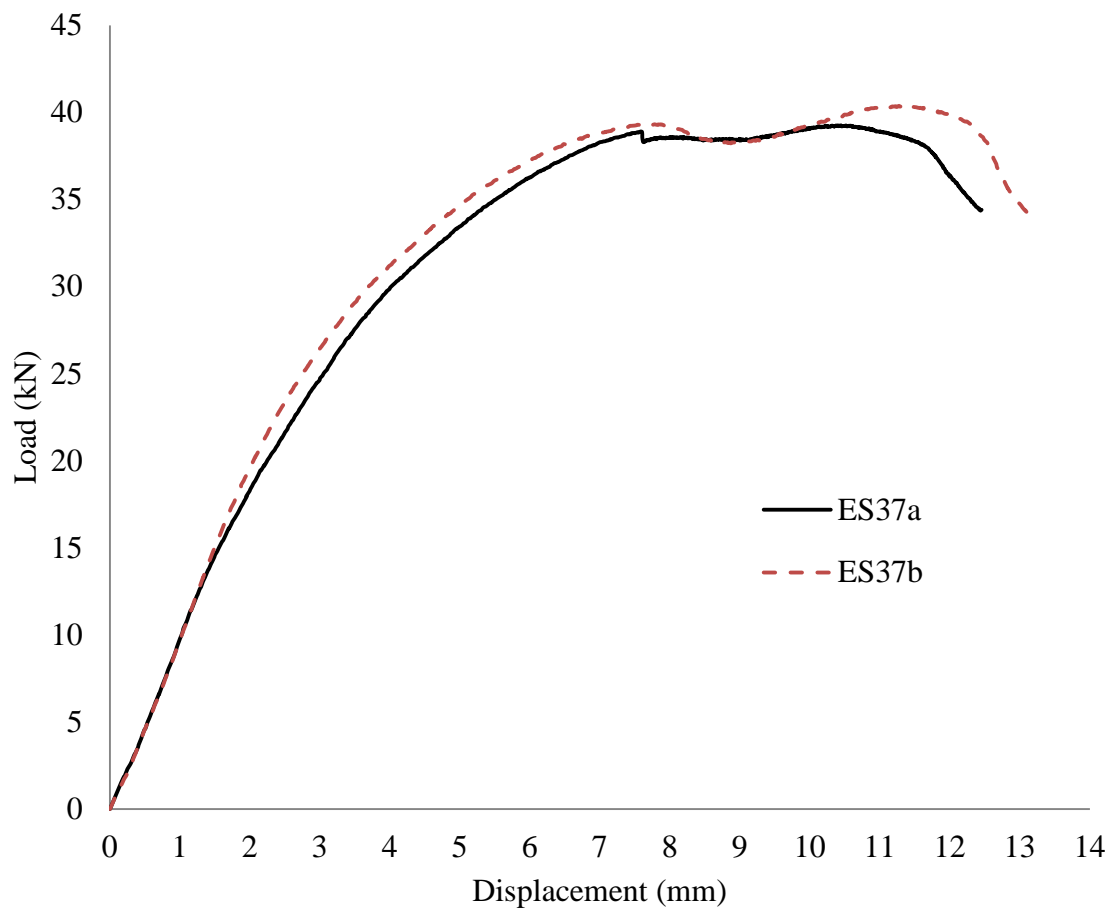


Figure 13 Load-displacement graphs of Specimens ES37a and ES37b

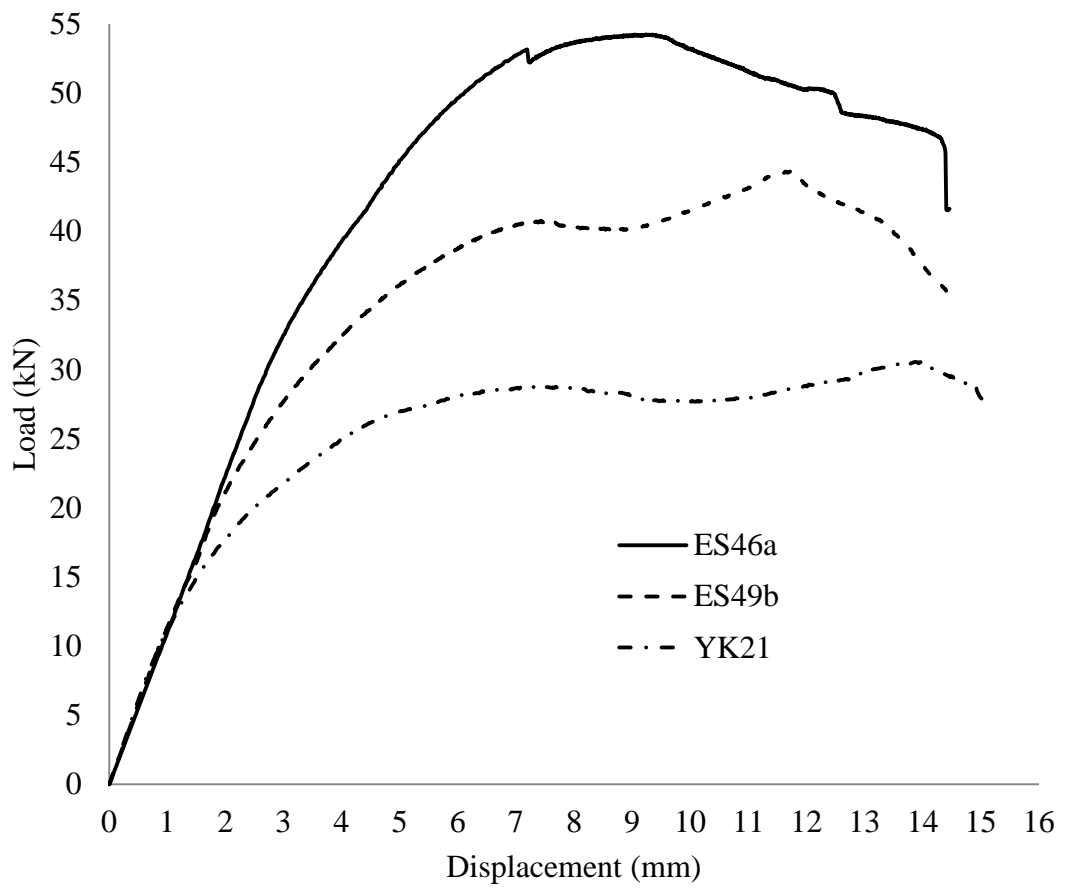


Figure 14 Load-displacement graphs of Specimens ES46a, ES49b and YK21

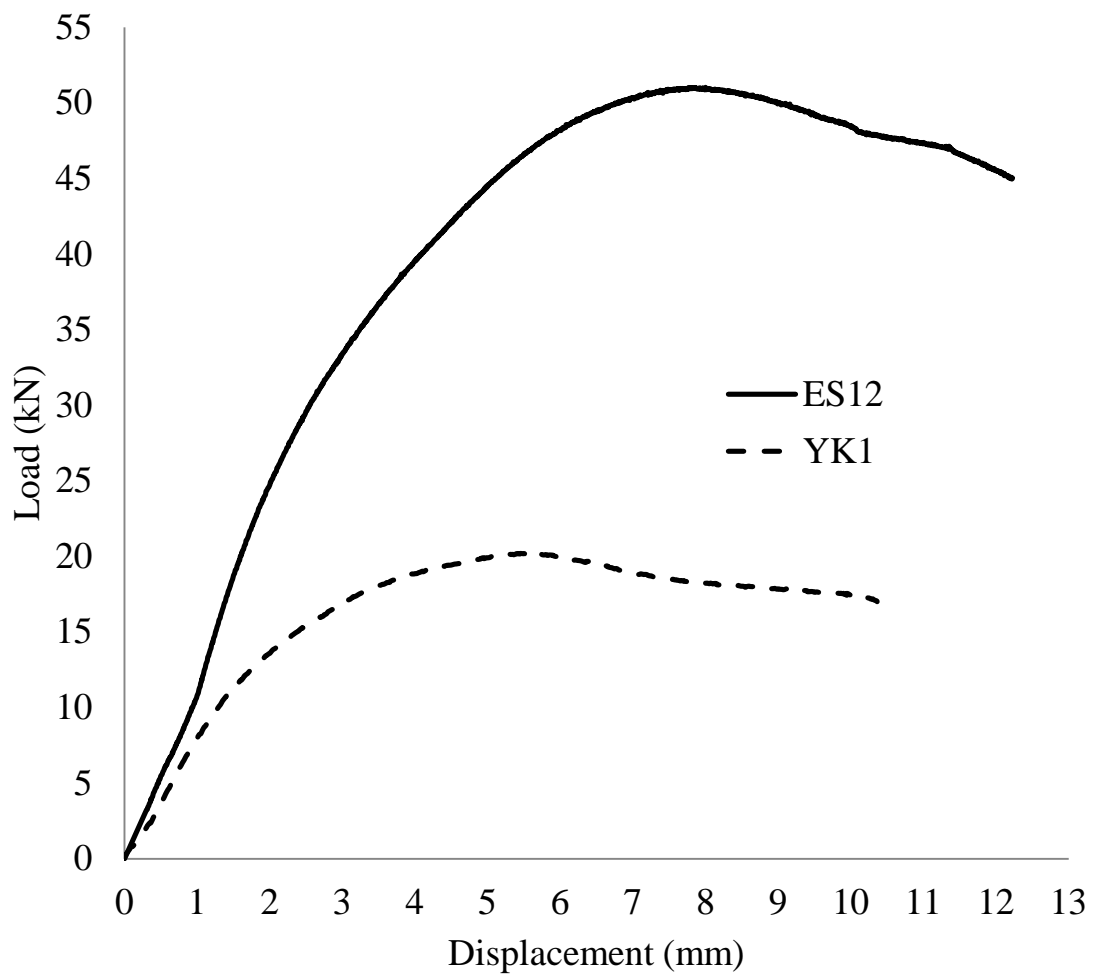


Figure 15 Load-displacement graphs of Specimens ES12 and YK1

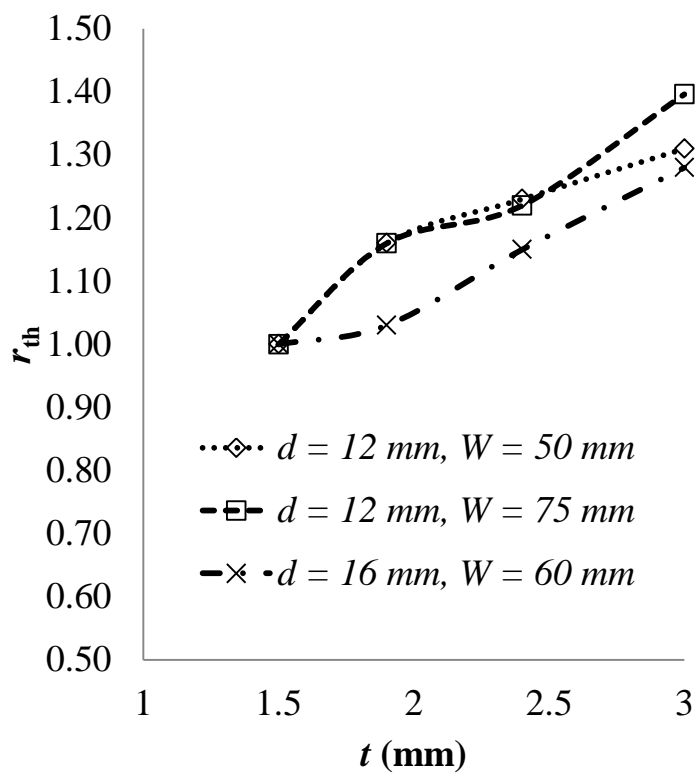


Figure 16 Exponential relationship between tilt bearing capacity and sheet thickness

Table 1 Bearing factor C in Section E3.3.1 (AISI 2012)

d/t	C
< 10	3.0
$10 \leq d/t \leq 22$	$4 - 0.1 d/t$
> 22	1.8

Table 2 Average material properties for G450 sheet steels

	t_{base} (mm)	F_y (MPa)	F_u (MPa)	F_u / F_y	ϵ_{15} (%)	ϵ_{25} (%)	ϵ_{50} (%)	ϵ_{u0} (%)
1.5 mm	1.48	555	590	1.06	21.5	16.3	12.0	6.9
1.9 mm	1.82	540	585	1.08	26.3	22.3	12.1	8.4
2.4 mm	2.36	535	580	1.08	31.0	23.8	16.3	8.9
3.0 mm	2.95	520	555	1.07	30.5	21.4	14.8	8.2

Table 3 Average material properties for G2 sheet steels

	t_{base} (mm)	F_y (MPa)	F_u (MPa)	F_u / F_y	ϵ_{15} (%)	ϵ_{25} (%)	ϵ_{50} (%)	ϵ_{u0} (%)
1.5 mm	1.45	320	400	1.25	55.2	45.9	37.7	24.5
2.4 mm	2.35	310	390	1.26	62.4	51.5	40.1	26.8

Table 4 Results of G450 specimens having 12-mm bolt with 2-mm hole clearance

Spec	W (mm)	t (mm)	r _{th}	P _t (kN)	P _t /P _p		
					(1)	(2)	(9)
ES31	50	1.5	Ref	14.9	0.63	0.57	0.90
ES51		1.9	1.16	21.0	0.73	0.66	0.97
ES53a		2.4	1.23	29.5	0.80	0.72	0.98
ES53b				28.0	0.76	0.68	0.92
ES33		3.0	1.31	36.6	0.83	0.75	0.93
ES35	60	1.5	Ref	15.3	0.65	0.58	0.88
ES55a		1.9	1.18	22.6	0.79	0.71	1.00
ES55b				21.5	0.75	0.67	0.95
ES57a		2.4	1.19	31.6	0.86	0.77	1.00
ES57b				31.3	0.85	0.76	0.99
ES37a		3.0	1.39	39.3	0.89	0.80	0.96
ES37b				40.4	0.91	0.82	0.99
ES39	70	1.5	Ref	17.1	0.73	0.65	0.96
ES41		3.0	1.38	44.3	1.00	0.90	1.05
ES47	75	1.5	Ref	17.5	0.74	0.67	0.97
ES59		1.9	1.16	24.8	0.86	0.78	1.05
ES61a		2.4	1.22	33.7	0.91	0.82	1.02
ES61b				33.0	0.89	0.80	0.99
ES49a		3.0	1.40	47.3	1.07	0.96	1.10
ES49b				44.3	1.00	0.90	1.03
ES44	100	1.5	Ref	19.0	0.81	0.73	0.99
ES63a		1.9	1.08	24.8	0.86	0.78	0.99
ES63b				25.1	0.87	0.79	1.00
ES71		2.4	1.13	33.8	0.91	0.82	0.96
ES70	120	1.9	Ref	24.9	0.87	0.78	0.96
ES69		2.4	1.10	35.4	0.96	0.86	0.97

Table 5 Results of G450 specimens having 16-mm bolt with 2-mm hole clearance

Spec	W (mm)	t (mm)	r_{th}	r_d	P_t (kN)	P_t/P_p		
						(1)	(2)	(9)
ES32	50	1.5	Ref	1.17	17.4	0.57	0.50	0.92
ES52		1.9	1.02	1.03	21.7	0.57	0.51	0.89
ES54a		2.4	1.16	1.10	32.3	0.66	0.59	0.94
ES54b					30.9	0.63	0.56	0.90
ES34		3.0	1.31	1.17	42.9	0.73	0.66	0.96
ES36	60	1.5	Ref	1.23	18.8	0.61	0.54	0.95
ES56		1.9	1.03	1.07	23.5	0.61	0.55	0.91
ES58a		2.4	1.15	1.08	34.8	0.71	0.64	0.96
ES58b					33.2	0.67	0.61	0.93
ES38		3.0	1.28	1.13	45.1	0.77	0.69	0.97
ES40	70	1.5	Ref	1.18	20.2	0.66	0.58	0.99
ES42		3.0	1.30	1.12	49.4	0.84	0.75	1.03
ES48a	75	1.5	Ref	1.32	22.9	0.75	0.66	1.11
ES48b					23.4	0.77	0.67	1.13
ES60		1.9	0.99	1.13	28.0	0.73	0.66	1.04
ES62a		2.4	1.07	1.15	38.3	0.78	0.70	1.01
ES62b					38.2	0.78	0.70	1.00
ES43		3.0	1.21		48.2	0.82	0.74	0.99
ES50				1.17	51.9	0.88	0.79	1.06
ES45	100	1.5	Ref	1.33	25.2	0.82	0.72	1.14
ES64a		1.9	0.99	1.14	26.9	0.70	0.63	0.94
ES64b					30.2	0.79	0.71	1.05
ES65a		2.4	1.08	1.18	40.2	0.82	0.73	1.00
ES65b					39.4	0.80	0.72	0.98
ES46a		3.0	1.22	N/A	54.2	0.92	0.83	1.04
ES46b					53.7	0.91	0.82	1.04
ES66a	120	1.9	Ref	1.17	28.6	0.75	0.67	0.96
ES66b					29.6	0.77	0.70	0.99
ES67a		2.4	1.13	1.21	43.1	0.87	0.79	1.03
ES67b					42.7	0.87	0.78	1.02
ES68		3.0	1.19	N/A	54.2	0.92	0.83	0.96

Table 6 Results of G450 specimens with 1-mm hole clearance

Spec	W (mm)	t (mm)	d (mm)	P_t (kN)	P_t/P_p		
					(1)	(2)	(9)
ES1a	50	1.5	12	16.8	0.71	0.64	1.01
ES1b				17.0	0.72	0.65	1.01
ES2a			16	19.1	0.62	0.55	1.01
ES2b				18.5	0.60	0.53	0.98
ES3a		3.0	12	39.8	0.90	0.81	1.01
ES3b				41.5	0.94	0.84	1.05
ES4a			16	44.3	0.75	0.68	0.99
ES4b				42.5	0.72	0.65	0.95
ES5a	60	1.5	12	17.2	0.73	0.66	0.99
ES5b				19.9	0.84	0.76	1.15
ES6a			16	21.3	0.70	0.61	1.08
ES7a		3.0	12	39.8	0.90	0.81	0.97
ES7b				43.6	0.99	0.89	1.06
ES8a			16	47.2	0.80	0.72	1.01
ES8b				47.1	0.80	0.72	1.01
ES9	70	1.5	12	18.2	0.77	0.69	1.01
ES10			16	23.5	0.77	0.67	1.15
ES11		3.0	12	45.2	1.02	0.92	1.07
ES12			16	51.0	0.87	0.78	1.06
ES13	75	1.5	12	18.5	0.78	0.71	1.02
ES14			16	23.8	0.78	0.68	1.14
ES15		3.0	12	44.8	1.01	0.91	1.04
ES16			16	54.6	0.93	0.83	1.12
ES17	100	1.5	12	19.2	0.81	0.73	1.00
ES18			16	24.6	0.80	0.70	1.12
ES19		3.0	12	46.1	1.04	0.94	1.02
ES20			16	57.2	0.97	0.87	1.10
ES21	120	1.5		24.3	0.79	0.70	1.06
ES22		3.0		57.4	0.97	0.88	1.06

Table 7 Results of G2 specimens with 2-mm hole clearance

Spec	W (mm)	t (mm)	d (mm)	Orientation	P _t (kN)	P _t /P _p		
						(1)	(2)	(9)
YK35	50	1.5	12	I	10.8	0.69	0.62	0.98
YK36				II	10.0	0.64	0.57	0.91
YK39			16	I	11.2	0.56	0.48	0.90
YK40				II	11.2	0.55	0.48	0.90
YK43	75		12	I	10.8	0.69	0.62	0.91
YK44				II	10.8	0.69	0.62	0.90
YK47			16	I	13.2	0.66	0.57	0.97
YK48				II	12.7	0.63	0.55	0.93
YK51	100		12	I	12.9	0.83	0.74	1.02
YK52				II	12.1	0.78	0.70	0.96
YK55			16	I	15.8	0.78	0.68	1.09
YK56				II	14.9	0.74	0.64	1.02
YK59	120		12	I	12.4	0.79	0.71	0.94
YK60				II	10.4	0.66	0.60	0.79
YK63			16	I	15.0	0.75	0.65	1.00
YK64				II	14.0	0.69	0.60	0.93
YK3	50	2.4	12	I	18.8	0.76	0.68	0.93
YK4				II	18.2	0.73	0.66	0.90
YK7			16	I	19.9	0.60	0.54	0.87
YK8				II	20.4	0.62	0.56	0.89
YK11	75		12	I	22.2	0.90	0.81	1.00
YK12				II	22.8	0.92	0.83	1.03
YK15			16	I	28.4	0.86	0.78	1.12
YK16				II	26.3	0.80	0.72	1.04
YK19	100		12	I	23.0	0.93	0.84	0.98
YK20				II	23.2	0.94	0.84	0.99
YK23			16	I	29.9	0.91	0.82	1.11
YK24				II	28.0	0.85	0.76	1.04
YK27	120		12	I	23.2	0.94	0.84	0.95
YK28				II	23.6	0.95	0.86	0.97
YK31			16	I	29.3	0.89	0.80	1.05
YK32				II	24.7	0.75	0.67	0.88

Table 8 Results of G2 specimens with 1-mm hole clearance

Spec	W (mm)	t (mm)	d (mm)	Orientation	P_t (kN)	P_t/P_p		
						(1)	(2)	(9)
YK33	50	1.5	12	I	10.4	0.66	0.60	0.94
YK34				II	10.7	0.69	0.62	0.98
YK37			16	I	10.5	0.52	0.45	0.84
YK38				II	13.3	0.66	0.57	1.07
YK41	75		12	I	13.9	0.89	0.80	1.16
YK42				II	12.0	0.77	0.69	1.00
YK45			16	I	15.3	0.76	0.66	1.11
YK46				II	15.7	0.78	0.68	1.15
YK49	100		12	I	14.2	0.91	0.82	1.12
YK50				II	14.5	0.93	0.84	1.15
YK53			16	I	13.5	0.67	0.58	0.93
YK54				II	16.8	0.83	0.72	1.15
YK57	120		12	I	13.9	0.89	0.80	1.06
YK58				II	13.9	0.89	0.80	1.06
YK61			16	I	16.5	0.82	0.71	1.10
YK62				II	15.0	0.75	0.65	1.00
YK1	50	2.4	12	I	20.3	0.82	0.74	0.99
YK2				II	19.9	0.80	0.72	0.97
YK5			16	I	20.4	0.62	0.56	0.88
YK6				II	19.7	0.60	0.54	0.85
YK9	75		12	I	24.9	1.02	0.92	1.13
YK10				II	23.7	0.96	0.86	1.07
YK13			16	I	27.1	0.82	0.74	1.06
YK14				II	28.1	0.85	0.77	1.11
YK17	100		12	I	25.1	1.02	0.91	1.07
YK18				II	24.5	0.99	0.89	1.04
YK21			16	I	30.6	0.93	0.83	1.13
YK22				II	29.3	0.89	0.80	1.09
YK25	120		12	I	25.3	1.02	0.92	1.04
YK26				II	25.4	1.02	0.92	1.04
YK29			16	I	30.6	0.93	0.84	1.10
YK30				II	30.6	0.93	0.83	1.09

Table 9 Results of single-shear single-row double bolted connections

Spec	W (mm)	t (mm)	d (mm)	P_t (kN)	P_t/P_p		
					(1)	(2)	(9)
EPD5	75	1.5	12	31.5	0.67	0.60	1.01
EPD9a	100			34.0	0.72	0.65	1.02
EPD9b				35.2	0.75	0.67	1.06
EPD10			16	41.9	0.68	0.60	1.11
EPD11		3.0	12	76.7	0.87	0.78	0.97
EPD12			16	86.3	0.73	0.66	0.97

Table 10 Results of independent researchers

Researchers	W (mm)	t (mm)	d (mm)	F_u (MPa)	P_t (kN)	P_t/P_p		
						(1)	(2)	(9)
Yu & Sheerah (2008)	101.6	0.92	6.4	375	5.18	1.06	0.96	1.09
					5.40	1.11	1.00	1.14
					5.09	1.04	0.94	1.07
					5.48	1.12	1.01	1.15
					5.02	1.03	0.93	1.06
					5.05	1.04	0.93	1.06
					8.16	1.06	0.95	1.02
					8.42	1.09	0.98	1.05
					8.12	1.05	0.95	1.02
					7.67	0.99	0.89	0.96
^a Casafont et al. (2006)	100	1.58	8	390	21.9	0.98	0.88	1.11
					12.3	0.93	0.84	0.99
Hoang et al. (2013)	42.5	2.00		365	12.3	0.93	0.84	0.99

a: Two bolts in a single row.

Table 11 Results for Yu & Sheerah (2008)'s specimens with 3.2 mm clearance on A325 bolt (oversized head)

<i>W</i> (mm)	<i>t</i> (mm)	<i>d</i> (mm)	<i>F_u</i> (MPa)	<i>P_t</i> (kN)	<i>P_t/P_p</i>		
					(1)	(2)	(9)
101.6	1.75	12.7	480	20.8	0.87	0.78	1.03
				22.0	0.91	0.82	1.09
				20.7	0.86	0.77	1.03

Table 12 Performance of alternative equations for 170 specimens

	Eqn (1), AISI (2012)	Eqn (2), ECS (2006)	Eqn (9), Proposed
<i>P_m</i>	0.82	0.74	1.01
COV	0.162	0.187	0.073