Nov 10th, 12:00 AM - 12:00 AM

Tilt Bearing Capacity of Single-Shear Bolted Connections without Washers

Mehmet E. Uz

Lip H. Teh

Wei-Wen Yu International Specialty Conference on Cold-Formed Steel Structures
Baltimore, Maryland, U.S.A, November 9 & 10, 2016

Tilt Bearing Capacity of Single-Shear Bolted Connections without Washers

Mehmet E. Uz[1] and Lip H. Teh[2]

Abstract

This paper examines the accuracy of design equations specified in the North American and European codes for cold-formed steel structures in determining the ultimate tilt bearing capacity of single-shear single-row bolted connections without washers in flat steel sheets. It points out that the code equations do not properly distinguish the tilt bearing failure mode from the conventional bearing failure mode, which is typical of double-shear connections and single-shear connections with washers. The tilt bearing capacity is affected by the width of the connected sheet, and its capacity does not vary linearly with either the sheet thickness or the bolt diameter. Based on the test results of 150 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. A resistance factor of 0.75 is recommended for use with the proposed equation for determining the ultimate tilt bearing capacity of single shear single-row bolted connections in cold-reduced steel sheets.

Introduction

The ultimate bearing capacity of a bolted connection is specified in Section E3.3.1 of AISI S100-12 (AISI 2012) and Table 7.4 of EN-1993-1-3:2006 (ECS

[1]Associate Research Fellow, ARC Research Hub for Australian Steel Manufacturing, Wollongong, NSW 2500, AUSTRALIA. Lecturer, Department of Civil Engineering, Adnan Menderes University, Aydin, PK:09100, Turkey.
[2]Associate Professor, School of Civil, Mining & Environmental Engineering, University of Wollongong, Wollongong, NSW 2500, AUSTRALIA.

2006). No fundamental distinction is made between double-shear and single-shear connections, although the AISI specification employs modification factors. However, the bearing failure mode typical of the inside sheet of a double-shear connection has a distinct mechanism from the tilt bearing failure mode of a single-shear connection without washers, as evident in Figure 1.

(a) Conventional (b) Tilt bearing

Figure 1 Different types of bearing failures

The conventional bearing failure shown in Figure 1(a) occurred on the downstream side of the bolt hole, while the tilt bearing failure in Figure 1(b) was due to the bolt head punching through the sheet on the upstream side during tilting. Bolt tilting and curling of the connected sheet occur due to the eccentricity of loading in a single-shear connection as illustrated in Figure 2.

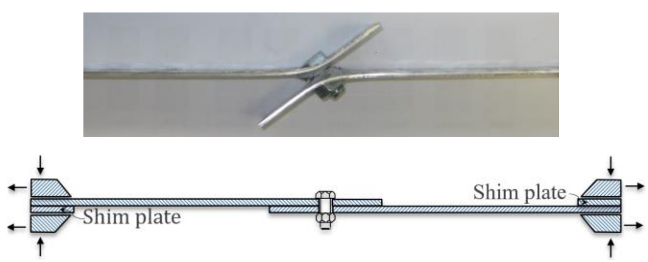

Figure 2. Single-shear connection subject to curling and bolt tilting

For a single-shear bolted connection with or without washers, there is another failure mode that was associated by some researchers with tilting and bearing. The failure mode is depicted in Figure 3, and was experienced by the specimens tested by Carril et al. (1994) and Casafont et al. (2006). Such a failure mode was called "localised tearing" by Rogers & Hancock (2000). Localised tearing was in the past mistaken to be the net section tension fracture mode (LaBoube 1988, Rogers & Hancock 2000), and is outside the scope of this paper.

Figure 3 Localised tearing (extracted from Casafont et al. (2006)

The authors did not detect evidence of tilt bearing failures among the specimens tested by Wallace & Schuster (2001). Figure 6(a) of their report shows a bearing failure on the downstream side of the bolt hole of a specimen without washers despite the presence of curling. Yu & Mosby (1981), who tested single-shear bolted connections in thin sheets, did not discuss the tilt bearing failure mode.

Rogers & Hancock (2000) did not define the failure mode that is due to the bolt head punching through the connected sheet on the upstream side of the bolt hole. The North American and the European guidelines on the testing of sheet steel connections (AISI 2008, ECCS 2009) describe five failure modes including the so-called "tilting and pull-out failure" mode, but do not mention the tilt bearing failure shown in Figure 1(b).

This paper presents the first ever systematic study on the tilt bearing capacities, which are due to the bolt head punching through the connected sheet on the upstream side of the bolt hole. It details how a nonlinear empirical equation for the tilt bearing capacity can be derived methodically without losing dimensional consistency. The design equation will be formulated based on the results of 150 G2 and G450 sheet steel specimens tested in the present work, and verified against independent test results of other researchers (Casafont et al. 2006, Yu & Sheerah 2008, Hoang et al. 2013) where the single-shear single-row bolted connection specimens are known to have failed by tilt bearing.

Interested readers may consult Teh & Uz (2014) for the conventional bearing failure mode, Teh & Gilbert (2012) for the net section tension fracture mode, Teh & Clements (2012) for the block shear failure mode, and Teh & Uz (2015) for the shear-out failure mode.

Equations for bearing capacity of single-shear bolted connection

Section E3.3.1 of AISI S100-12 (AISI 2012) specifies the bearing capacity per bolt of a single-shear bolted connection without washers to be

$$P_b = 0.75 C\, d\, t F_u \tag{1}$$

in which d is the bolt diameter, t is the sheet thickness and F_u is the material tensile strength. The bearing factor C depends on the ratio of the bolt diameter d to the sheet thickness t, as given in Table E3.3.1-1 of the specification.

The "modification factor" of 0.75 in Equation (1) is supposed to differentiate a single-shear connection from a double-shear one (1.33), and accounts for the absence of washers (1.00).

The Australasian standard (SA/SNZ 2005) adopts Equation (1). On the other hand, the European code (ECS 2006) does not even make a distinction between single-shear and double-shear connections, and does not consider the benefit of washers. For the specimens tested in the present work, in which the end distance was invariably more than 3 times the bolt diameter, the European code specifies the bearing strength per bolt to be

$$P_b = 2.5 k_t \, d \, t F_u \tag{2}$$

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

$$k_t = \frac{0.8t + 1.5}{2.5}; \quad 0.75\,\text{mm} \le t \le 1.25\,\text{mm} \tag{3}$$

For Equation (3) to be valid (not necessarily accurate), the sheet thickness t must be measured in millimetres since the two constants are dimensionless.

The width of the connected sheet is likely to affect the tilt bearing capacity as the resistance to curling increases with increasing sheet width, yet this parameter is absent in both code equations. In the present work, the tilt bearing capacity per bolt of a single-shear single-row bolted connection is expressed as

$$P_b = C_{\text{tb}} \, d^b \, t^a W_n^c \, F_u \tag{4}$$

in which W_n is the sheet width that is net of the bolt hole diameter. For single-row bolted connections having more than one bolt, the net sheet width W_n is equal to the total net sheet width divided by the number of bolts.

The ultimate tilt bearing coefficient C_{tb} and the exponential terms a through c would be determined through analyses of the present test results, and verified against independent test results of other researchers (Casafont et al. 2006, Yu & Sheerah 2008, Hoang et al. 2013).

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

$$c = 2 - (a + b) \tag{5}$$

Test materials

The G450 and G2 sheet steel materials used in the present laboratory tests, which have trade names GALVASPAN® and GALVABOND®, respectively, were manufactured by Bluescope Steel Australia. G450 sheet steel is a structural grade covered by the Australasian standard (SA/SNZ 2005) for which the nominal yield stress and tensile strength may be fully utilised in structural design calculations. The average yield stresses F_y, tensile strengths F_u and elongations at fracture over 15 mm, 25 mm and 50 mm gauge lengths ε_{15}, ε_{25} and ε_{50}, and uniform elongation outside the fracture ε_{uo} of the steel materials as obtained from 12.5 mm wide tension coupons are shown in Tables 1 and 2 for the G450 and G2 sheet steels, respectively.

Table 1 Average 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_{uo} (%)
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 2 Average 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_{uo} (%)
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

Specimen configurations and test arrangements

All specimens tested in the present work were single-shear single bolted connections, as illustrated in Figure 2. The distance between each bolt and the downstream end was at least 50 mm to prevent the shear-out failure mode.

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

In order to ascertain the effect of sheet width, for each thickness of the G450 sheet steels, the widths were 50, 60, 70, 75, 100 and 120 mm. These values represent the range that may be covered by one bolt in cold-formed steel constructions. The derived equation would be verified against the test results of Yu & Sheerah (2008) involving a ratio *W*/*d* close to 16.

Two bolt sizes commonly used for structural connections in G450 sheet steels, 12 and 16 mm, were used. The proposed equation would be verified against the test results of Yu & Sheerah (2008) involving 6.4 mm bolts, and those of Casafont et al. (2006) and Hoang et al. (2013) involving 8 mm bolts.

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

A total of sixty four specimens composed of G2 sheet steel would be tested in light of the finding of Teh & Uz (2014) regarding the effect of material ductility on the bearing capacity of double-shear connections. The G2 specimens also provided an opportunity to investigate the effect of the orientation of bolt head and nut on the tilt bearing capacity. The two orientations are shown in Figure 4.

(a) (b)

Figure 4 Two orientations of bolt head and nut:
(a) Orientation I; (b) Orientation II

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

Tables 3 and 4 lists the geometric dimensions and ultimate test loads of G450 specimens that had a nominal bolt hole clearance of 2 mm, for 12-mm and 16-mm bolts, respectively. The variable P_t in the tables denotes the ultimate test load, while P_p is the tilt bearing capacity predicted by the equations.

Table 3 Test 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

The variations of the tilt bearing capacity with the sheet thickness were checked against 12 groups of specimens shown in Tables 3 and 4. The normalised capacity ratio r_{th} shown in the tables were calculated from

$$r_{\text{th}} = \frac{P_t F_{u\text{ref}}\, t_{\text{ref}}}{P_{t\text{ref}}\, F_u\, t} \tag{6}$$

Table 4 Test 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

The exponential term *a* in Equation (4) should satisfy

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

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

$$a = 1 + {}^{i}\!/_{j} \tag{8}$$

in which i and j are positive integers.

It was found that using $a = {}^{4}/_{3}$ simulated the relationship between the tilt bearing capacity and the sheet thickness quite well.

The variable r_{d} in Table 4 denotes the ratio between the ultimate test load of a 16-mm bolt specimen and that of the corresponding 12-mm bolt specimen. The average value of r_{d} is 1.16. Using b = ½ in Equation (4) would give a ratio of 1.15. The exponential term b is therefore taken to be ½, meaning that the tilt bearing capacity varies with the square root of the bolt diameter.

Having determined the exponential terms a and b to be ${}^{4}/_{3}$ and ½, respectively, the exponential term c was computed from Equation (5) to be ${}^{1}/_{6}$.

Ultimate tilt bearing coefficient and verification

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

Tables 6 and 7 list the geometric dimensions and ultimate test loads of G2 specimens that had nominal bolt hole clearances of 2 mm and 1 mm, respectively. By comparing the professional factors in these tables against those in Tables 3 through 5 for comparable specimens, it can be concluded that the significantly different levels of material ductility between G2 and G450 sheet steels, as evident from Tables 1 and 2, did not affect the tilt bearing capacities.

The results shown in Tables 6 and 7 also indicate that the orientations of the bolt head and nut did not have significant effect on the tilt bearing capacity, although there was some 5% difference on average for the specimens in Table 6.

Table 5 Test 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

Having established that variations in bolt hole clearances, material ductility, and bolt head/nut orientation do not have meaningful effects on the tilt bearing capacity of single-shear single-row bolted connections, the ultimate tilt bearing coefficient C_{tb} in Equation (4) was determined to be 2.65 based on the ultimate test loads of 150 specimens listed in Tables 3 through 7 and the exponential terms a, b and c computed in the preceding section. Equation (4) becomes

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

The professional factors of Equation (9) are given in Tables 3 through 8, along with those of Equations (1) and (2).

Table 6 Test 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)
YK 35	50	1.5	12	I	10.8	0.69	0.62	0.98
YK 36				II	10.0	0.64	0.57	0.91
YK 39			16	I	11.2	0.56	0.48	0.90
YK 40				II	11.2	0.55	0.48	0.90
YK 43	75		12	I	10.8	0.69	0.62	0.91
YK 44				II	10.8	0.69	0.62	0.90
YK 47			16	I	13.2	0.66	0.57	0.97
YK 48				II	12.7	0.63	0.55	0.93
YK 51	100		12	I	12.9	0.83	0.74	1.02
YK 52				II	12.1	0.78	0.70	0.96
YK 55			16	I	15.8	0.78	0.68	1.09
YK 56				II	14.9	0.74	0.64	1.02
YK 59	120		12	I	12.4	0.79	0.71	0.94
YK 60				II	10.4	0.66	0.60	0.79
YK 63			16	I	15.0	0.75	0.65	1.00
YK 64				II	14.0	0.69	0.60	0.93
YK 3	50	2.4	12	I	18.8	0.76	0.68	0.93
YK 4				II	18.2	0.73	0.66	0.90
YK 7			16	I	19.9	0.60	0.54	0.87
YK 8				II	20.4	0.62	0.56	0.89
YK 11	75		12	I	22.2	0.90	0.81	1.00
YK 12				II	22.8	0.92	0.83	1.03
YK 15			16	I	28.4	0.86	0.78	1.12
YK 16				II	26.3	0.80	0.72	1.04
YK 19	100		12	I	23.0	0.93	0.84	0.98
YK 20				II	23.2	0.94	0.84	0.99
YK 23			16	I	29.9	0.91	0.82	1.11
YK 24				II	28.0	0.85	0.76	1.04
YK 27	120		12	I	23.2	0.94	0.84	0.95
YK 28				II	23.6	0.95	0.86	0.97
YK 31			16	I	29.3	0.89	0.80	1.05
YK 32				II	24.7	0.75	0.67	0.88

Table 7 Test 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)
YK 33	50	1.5	12	I	10.4	0.66	0.60	0.94
YK 34				II	10.7	0.69	0.62	0.98
YK 37			16	I	10.5	0.52	0.45	0.84
YK 38				II	13.3	0.66	0.57	1.07
YK 41	75		12	I	13.9	0.89	0.80	1.16
YK 42				II	12.0	0.77	0.69	1.00
YK 45			16	I	15.3	0.76	0.66	1.11
YK 46				II	15.7	0.78	0.68	1.15
YK 49	100		12	I	14.2	0.91	0.82	1.12
YK 50				II	14.5	0.93	0.84	1.15
YK 53			16	I	13.5	0.67	0.58	0.93
YK 54				II	16.8	0.83	0.72	1.15
YK 57	120		12	I	13.9	0.89	0.80	1.06
YK 58				II	13.9	0.89	0.80	1.06
YK 61			16	I	16.5	0.82	0.71	1.10
YK 62				II	15.0	0.75	0.65	1.00
YK 1	50	2.4	12	I	20.3	0.82	0.74	0.99
YK 2				II	19.9	0.80	0.72	0.97
YK 5			16	I	20.4	0.62	0.56	0.88
YK 6				II	19.7	0.60	0.54	0.85
YK 9	75		12	I	24.9	1.02	0.92	1.13
YK 10				II	23.7	0.96	0.86	1.07
YK 13			16	I	27.1	0.82	0.74	1.06
YK 14				II	28.1	0.85	0.77	1.11
YK 17	100		12	I	25.1	1.02	0.91	1.07
YK 18				II	24.5	0.99	0.89	1.04
YK 21			16	I	30.6	0.93	0.83	1.13
YK 22				II	29.3	0.89	0.80	1.09
YK 25	120		12	I	25.3	1.02	0.92	1.04
YK 26				II	25.4	1.02	0.92	1.04
YK 29			16	I	30.6	0.93	0.84	1.10
YK 30				II	30.6	0.93	0.83	1.09

Equation (9) was checked against the test results of independent researchers where the specimens failed by tilt bearing due to the bolt head punching through the connected sheet on the upstream side of the bolt hole, and where the nominal hole diameter clearance did not exceed 2 mm. Yu & Sheerah (2008) tested 12 such specimens with a diameter clearance of 1.5 mm. Casafont et al. (2006) tested single-shear single-row bolted connections having two bolts each. From the photographs provided in their paper, most of the specimens appear to have failed in the localised tearing mode depicted in Figure 3. However, one specimen having a clearance of 1 mm, shown in Figs. 31 and 32 of their paper, failed in tilt bearing due to the bolt head punching through the connected sheet on the upstream side of the bolt hole. Hoang et al. (2013) tested one specimen only, with a clearance of 0.5 mm. All these test results are included in Table 8.

Table 8 Results of independent researchers

Researchers	***W*** **(mm)**	***t*** **(mm)**	***d*** **(mm)**	F_u **(MPa)**	P_t **(kN)**	P_t/P_p **(1)**	**(2)**	**(9)**
	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
Yu & Sheerah					5.05	1.04	0.93	1.06
(2008)		1.12		485	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
					7.96	1.03	0.93	1.00
					8.11	1.05	0.94	1.01
Casafont et al. (2006)	100	1.58	8	390	21.9	0.98	0.88	1.11
Hoang et al. (2013)	42.5	2.00		365	12.3	0.93	0.84	0.99

It transpired that, for the specimens listed in Table 8, Equations (1) and (9) give professional factors that are relatively close to each other.

Resistance factor

The overall professional factor P_t/P_p given by Equation (9) for the 164 specimens listed in Tables 3 through 8 is 1.01, with a coefficient of variation equal to 0.074. In order to achieve the target reliability index β_0 of 3.5 in the LRFD approach, a resistance factor of 0.73 was computed according to Section F1.1 of the North American specification (AISI 2012). It is recommended that a resistance factor ϕ equal to 0.75 (rounded to the nearest 0.05) be used in conjunction with Equation (9) for determining the ultimate tilt bearing capacity of a single-shear single-row bolted connection in flat steel sheets.

Concluding remarks

This paper has presented the first ever systematic study on the tilt bearing capacities of single-shear single-row bolted connections in flat sheets. The tilt bearing failure is due to the bolt head punching through the connected sheet on the upstream side of the bolt hole during tilting, while the conventional bearing failure takes place on the downstream side. It has been found that the tilt bearing capacity is not affected by the variation in material ductility.

The tilt bearing capacity varies nonlinearly with the sheet thickness with an exponent equal to $^{4}/_{3}$, and is proportional to the square root of the bolt diameter.

The proposed design equation for the ultimate tilt bearing capacity of a single-shear single-row bolted connection in flat steel sheet, which includes the sheet width as a parameter, is dimensionally consistent. It is reasonably accurate for 164 specimens tested by the authors and other researchers, comprising specimens having sheet thicknesses ranging from 0.92 mm to 3.0 mm and bolt diameters ranging from 6.4 mm to 16 mm with hole clearances ranging from 0.5 mm to 2.0 mm. The tested ratios of sheet width to bolt diameter ranged from 3 to 16. The accuracy of the proposed design equation has not been found to be significantly affected by the orientations of the bolt head and nut.

It is recommended that a resistance factor of 0.75 be used in conjunction with the proposed design equation in the LRFD approach of the North American specification for the design of cold-formed steel structures.

Acknowledgment

The authors would like to thank the Australian Research Council for funding this research through the ARC Research Hub for Australian Steel Manufacturing under the Industrial Transformation Research Hubs scheme (Project ID: IH130100017). The steel materials used in the present laboratory tests were supplied by Messrs. Trevor Clayton and John Kralic of Bluescope.

References

AISI (2008) *Test Methods for Mechanically Fastened Cold-Formed Steel Connections*, ANSI/AISI S905-08, American Iron and Steel Institute, Washington, DC.

AISI (2012) *North American Specification for the Design of Cold-formed Steel Structural Members 2012 Edition*, American Iron and Steel Institute, Washington DC.

Carril, J. L., LaBoube, R. A., and Yu, W. W. (1994) *Tensile and bearing capacities of bolted connections*, First Summary Report, Civil Engineering Study 94-1, University of Missouri-Rolla, Rolla, MO.

Casafont, M., Arnedo, A., Roure, F., and Rodriguez-Ferran, A. (2006) “Experimental testing of joints for seismic design of lightweight structures. Part 2: Bolted joints in straps.” *Thin-Walled Struct.*, 44, 677-691.

ECCS (2009) *The Testing of Connections with Mechanical Fasteners in Steel Sheeting and Sections*, ECCS TC7 TWG 7.10, European Commission for Constructional Steelwork, Mem Martins, Portugal.

ECS (2006) *Eurocode 3: Design of steel structures, Part 1-3: General rules – Supplementary rules for cold-formed members and sheeting*, EN 1993-1-3:2006, European Committee for Standardisation, Brussels, Belgium.

Hoang, T. D., Herbelot, C., Imad, A., and Benseddiq, N. (2013) “Numerical modelling for prediction of ductile fracture of bolted structure under tension shear loading.” *Finite Elements in Analysis and Design*, 67, 56-65.

LaBoube, R.A. (1988). "Strength of bolted connections: Is it bearing or net section?", *Proc., 9th Int. Specialty Conf. Cold-Formed Steel Structures*, St Louis, MO, 589-601.

Rogers, C. A., and Hancock, G. J. (2000) “Failure modes of bolted-sheet-steel connections loaded in shear.” *J. Struct. Eng.*, 126 (3), 288-296.

SA/SNZ (2005) *Cold-Formed Steel Structures*, AS/NZS 4600:2005, Standards Australia/Standards New Zealand, Sydney, Australia.

Teh, L. H., and Gilbert, B. P. (2012) “Net section tension capacity of bolted connections in cold-reduced steel sheets.” *J. Struct. Eng.*, 138 (3), 337-344.

Teh, L. H., and Clements, D. D. A. (2012) “Block shear capacity of bolted connections in cold-reduced steel sheets,” *J. Struct. Eng.*, 138 (4), 459-467.

Teh, L. H., and Uz, M. E. (2014) “Effect of loading direction on the bearing capacity of cold-reduced sheet steels.” *J. Struct. Eng.*, 140 (12), 06014005.

Teh, L. H. and Uz, M. E. (2015) “Ultimate shear-out capacity of structural steel bolted connections.” *J. Struct. Eng.*, 141 (6), 04014152.

Yu, C., and Sheerah, I. (2008) *Cold-formed Steel Bolted Connections Without Washers on Oversized and Slotted Holes*, Research Report RP08-11, Committee on Specifications for the Design of Cold-formed Steel Structural Members, American Iron and Steel Institute, Washington, DC.

Yu, W. W., and Mosby, R. L. (1981) *Bolted connections in cold-formed steel structures*, Final Report, Civil Engineering Study 81-1, University of Missouri-Rolla, Rolla, MO.

Wallace, J. A., and Schuster, R. M. (2001) *Testing of bolted cold formed steel connections in bearing (with and without washers)*, Research Report RP01-4, Committee on Specifications for the Design of Cold-formed Steel Structural Members, American Iron and Steel Institute, Washington, DC.