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Effect of plate properties on shear strength of bolt group in single plate connection

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Abstract. A single plate shear connection, or shear tab, is a very popular shear connection due to its merit in ease of construction and material economy. However, problems in understanding the connection behavior, both in terms of strength and ductility, have been well-documented. Suggestions or design model for single plate connections in AISC Design Manual have been altered several times, with the latest edition settling down to giving designers pre-calculated design strength tables if the connection details agree with given configurations. Results from many full-scale tests and finite element models in the past suggest that shear strength of a bolt group in single plate shear connections might be affected by yield strength of plate material; therefore, this research was aimed to investigate and clarify effects of plate yield strength and thickness on shear strength of the bolt group in the connections, including the validity of using a plate thickness/bolt diameter ratio (t_p/d_b) in design, by using finite element models. More than 20 models have been created by using ABAQUS program with 19.0- and 22.2-mm A325N bolts and A36 and Gr.50 plates with various thicknesses. Results demonstrated that increase of plate thickness or plate yield strength, with the t_p/d_b ratio remained intact, could significantly reduce shear strength of the bolt group in the connection as much as 15 percent. Results also confirmed that the t_p/d_b ratio is a valid indicator to be used for guaranteeing strength sufficiency. Because the actual ratio recommended by AISC Design Manual is t_p/d_b + 1.6 (mm) for connections with a number of bolts less than six and plate yield strength in construction is normally higher than the nominal value used in design, it is proposed that shear strength of a bolt group in single plate connections with a number of bolts equal or greater than seven be reduced by 15 percent and the t_p/d_b ratio be limited to 0.500.

Keywords: ABAQUS; finite element; shear strength of bolt group; single plate shear connection

1. Introduction: Background of single plate connections

1.1 Overview of design models from AISC design manuals

A single plate shear connection, or shear tab, is a very popular shear connection due to its merit in ease of construction and material economy. The connection itself consists of a steel plate of any grades, bolted on the beam side with either A325 or A490 bolts and welded on the support side,

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generally with either E60 or E70 material (Fig. 1). Design of the connection according to AISC Manual of Steel Construction (1999) was categorized into two types depending on the support condition: a rigid support type when the connection is attached to a strong support such as a column; and a flexible support type when the connection is attached to a less rigid support such as a girder, which tends to rotate. However, design model in the most recent Manual (AISC 2010) suggests that the configuration of the connection be arranged into two groups: the conventional and the extended configurations (Hewitt 2008).

For a single plate connection to be classified as a conventional one, the connection must have a number of bolts no more than 12 aligned in a single column format with the a-distance, which is the distance measured from the weld line to the bolt line, not greater 88.9 mm. Any single plate connections with configurations different from these criteria will be classified as extended. Pre-calculated design strength of the conventional configuration, based on relevant limit states of plate shear yielding, plate shear rupture, block shear, bearing/tear-out, weld shear, and bolt shear rupture and $t_p \leq d_b/2 + 1/16$ (mm) limit, was once offered in the table format in AISC Design Manual (AISC 2005). Shear strength of a bolt group in single plate shear connections have been modified several times. The bolt group was considered to have eccentricity as a function of the number of bolts (n) and a-distance (AISC 1995). After the connection has been categorized into conventional and extended configurations in the 13th edition manual, the bolt group in conventional configuration would not have eccentricity when the number of bolts is not greater than 9 (AISC 2005). However, in the latest Manual (AISC 2010), eccentricity of the bolt group has been reinstated into the conventional configuration as a function of the a-distance as follows

$$\text{When } n \leq 5, \quad e = a/2 \quad \text{for both standard and shot - slotted holes} \tag{1}$$

$$\text{When } n > 5, \quad e = a \quad \text{for standard holes} \tag{2}$$

The number of effective bolts based on coefficient C is used directly without any further modification unlike the previous version. In addition, the plate thickness limit has also been reintroduced using the same t_p/d_b ratio as follows

$$\text{When } n \leq 5, \quad t_p \leq d_b/2 + 1.6 \tag{3}$$

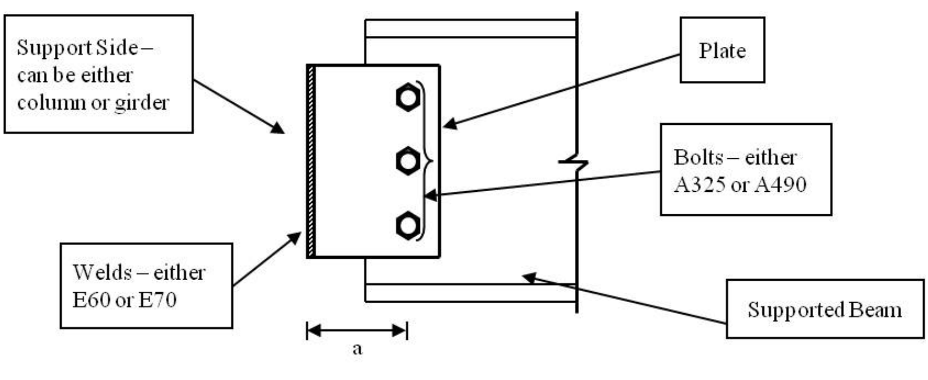


Fig. 1 General configuration of single plate shear connection

$$\text{When } n > 5,\ \ t_p \le d_b / 2 - 1.6 \tag{4}$$

For any extended single plate connections, eccentricity must be taken into account along with other considerations such as maximum plate thickness to ensure that moment capacity of the plate does not exceed that of the bolt group, and plate flexural strength with von-Mises shear reduction (AISC 2010).

For unstiffened extended single plate shear connections, Muir and Hewitt (2009) proposed a design model stating that strength of the bolt group be designed using the instantaneous center of rotation method with eccentricity equal to the distance from the support to the center of the bolt group, and 20% strength reduction may be omitted.

1.2 Overview of past research

Richard *et al*. (1980) were among the first researchers to start studying behaviors of single plate connections by carrying out five full-scale tests of connections with A325N bolts and A36 plates and comparing the results with finite element analysis. The research followed suggestions by Lipson (1968), and resulted in the proposed design model involved properties of the beam with strength of the connection. One important recommendation from his work is the limit of plate thickness to ensure ductility of the connection, which is

$$t_p \le d_b / 2 \tag{5}$$

Astaneh *et al*. (1988, 1989, 1993) conducted a number of full-scale tests of connections with A325N bolts and A36 plates and concluded that a bolt group and welds in the connection be designed for eccentricity by using the formulas which were the functions of the a-distance and the number of bolts in the connection. Astaneh and Nader (1989, 1990), based on the research of a Tee connection, also recommended the new limit of plate thickness to ensure ductility of the connection, which is

$$t_p \le d_b / 2 + 1.6 \ \ \text{(mm)} \tag{6}$$

This limitation was dropped from the 13th Manual (AISC 2005), but available strength shown is in accordance with it, i.e., no values are available for conventional configurations with plate thickness exceeds the number given by this inequality. However, in the 14th Manual (AISC 2010), the limitations are reinstate with modification as discussed in Section 1.1.

Sarkar (1992) later conducted full-scale tests of 2-, 4-, and 6-bolt connections using A36 steel plates and beams. Results showed that shear strength of the bolt group in 6-bolt connections was significantly lower than its nominal shear strength. However, it should be noted that yield strength of plate used, even though marked as A36, was relatively high (average yield equal to 327 MPa).

Following the design model proposed by Astaneh, a number of research have been conducted to verify the validity of the formulas. Ashakul and Murray (2004, 2006a, b) have performed finite element analysis on single plate shear connections up to seven bolts with various a-distances and A36 ($F_y = 248$ MPa) and Gr.50 ($F_y = 345$ MPa) plates. It was concluded that a-distance did not affect bolt shear strength in the connection, but plate yield strength might affect both shear strength and ductility of the connection.

Crocker and Chambers (2004a, b) have investigated rotation and deflection demands of single plate connections induced by seismic load. Three full-scale tests of three, four, and six A325N bolts with A36 plate were carried out. The t_p/d_b ratio in each test was exactly 0.500. Results have shown that these connections were ductile and the topmost bolts, which underwent most movement, displaced close to the displacement reported by Crawford and Kulak (1971). They also developed equations relating a rotation demand of the beam to a deflection demand, i.e., horizontal movement of the topmost bolt of the connection. By using the mean value of the position of connection neutral axis obtained from the tests, which was 0.63, the equations can be used to calculate the rotation capacity of single plate connections including predicting failure of the bolts. Another equation is also available for calculating rotation release from using a short-slotted hole. Experimental data from their work has later been used by Yim and Krauthammer (2012) to develop the mechanical models of single plate connections.

Creech (2005) carried out 10 full-scale tests to investigate design models used in AISC 3rd Edition Design Manual (2001), Great Britain, Australia, and other models proposed by many studies. It was concluded that the design model hitherto adopted by AISC was the most conservative, and that design of the connection could be performed by neglecting the effect of eccentricity on bolt shear strength if the bolt group action was excluded.

Metzger (2006) recently conducted full-scale tests of four conventional and four extended single plate connections with A325N bolts and Gr.50 plates, which had yield strength equal to 469 MPa, designed in accordance with the 13th Edition Manual (AISC 2005). It was found that shear strength of the bolt groups in the connections was lower than their nominal shear strength in 5- and 7-bolt connections by 5.8 and 8.9 percent, respectively.

Study of progressive collapse of steel structures has been under great interest of late. The study involves simulating a system of structures by utilizing mechanical models or mathematical realization to represent connections in the models. The fiber model to represent single plate shear connections has been developed by Weigand and Berman (2009) by considering a tributary bearing area on the plate along with information from tests on single bolt bearing by Rex and Easterling (2003). It should be noted that these models were based on one dimensional displacement of the bolt.

Garlock and Selamat (2010) have studied behavior of single plate shear connections under the event of fire by using Finite Element Model ABAQUS and suggested use of standard oversized holes in beam web to accommodate free movement of the beam during cooling stage (Selamat and Garlock 2009). This suggestion not only does not affect the governing limit state of single plate shear connections, but might also increase overall rotational capacity of the connection.

Marosi *et al*. (2011) have carried out eight full-scale tests of single plate connections with multiple rows of bolts. Plates with various thicknesses with F_y equal to 300 MPa were used along with 19, 22.2, and 25.4 mm A325 bolts. Tests with one bolt row generally had t_p/d_b ratios much lower than the AISC recommended value; whereas some tests with multiple rows of bolts had t_p/d_b ratios greater than the AISC recommended value. Results have shown that all tests had satisfactory rotation. It is also worth mentioning that, even though the purpose of increasing the number of bolts is to increase the tension capacity of the connection, shear strength of the connection was not increased effectively, especially when the number of rows was increased from two to three.

Tension capacity of single plate connections according to International Building Code (IBC), as well as NYC Building Code, was observed by Geschwinder and Gustafson (2010). IBC states that beams and girders shall have axial tensile strength equal to required vertical shear strength (ASD) or 2/3 of required vertical shear strength (LRFD), but not less than 44.5 kN. This demand is also directly transferred onto connections. Investigation has shown that single plate connections with conventional configurations satisfy these requirements.

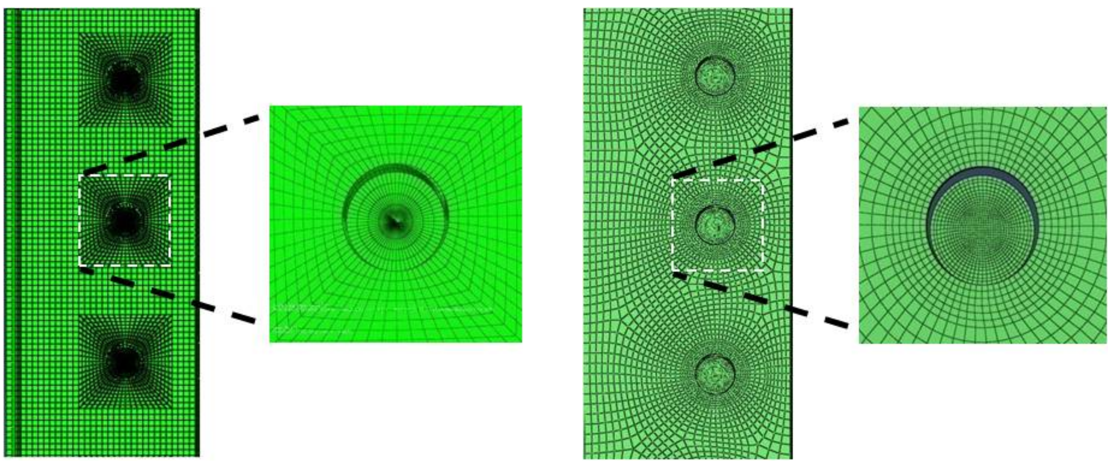
2. Finite element models

2.1 Model construction

Finite element program ABAQUS (2007) was employed exclusively throughout the research (Khampha 2010.) In constructing the finite element models, all the limit states listed in Section 1 have been checked to ensure that bolt shear rupture would be the governing limit state of every model. Types of element used in the models were C3D20 in the region where stress was high and contact problems were complicated, i.e., bolts, plates and areas around bolt holes in the beams, and C3D10M for the remaining portion of the beams.

A simple beam in the simulation was modeled by taking advantage of symmetry and employing a shear-release support at mid span. The beam was braced throughout the length to prevent lateral torsional buckling and was loaded with a uniformly distributed load, which was applied as pressure on top of top flange elements aligned with web elements. A plate was constrained in all directions at the supporting end to simulate welds, and bolts were constrained in the direction parallel to their longitudinal axes to bypass the construction of the nut, which would introduce a complex contact problem without being necessary. The STATIC method of analysis, which is a regular option in ABAQUS, was used. The NLGEOM option was used to include the nonlinear effect of the geometry. The rate of loading was linear.

Mesh refinement used in this research was based on the optimal study on a single plate by Ashakul (2004). Models of a 9.50 mm × 108 mm × 229 mm plate, which was used in Astaneh's experiments, was simulated using a relatively fine mesh with an element size of 6.40 mm × 6.40 mm × 4.80 mm and a very refined mesh with an element size of 3.20 mm × 3.20 mm × 4.80 mm. The model with the fine mesh yields a slightly stiffer result, while the result from the model with the very refined mesh was very close to the von Mises yield criterion. The model with fine mesh also demonstrated much smaller rotation in the inelastic region. As a result, the very fine mesh model was selected for all simulations by Ashakul. A more refined mesh was deemed unnecessary



(a) Mesh by Ashakul (2004)

(b) Improved mesh

Fig. 2 Improved mesh using CAE

since the result given by the very fine mesh was close to the calculated value from the shear yielding force. All models in this research were created with the same refinement by using ABAQUS/CAE (Computer-Aided Engineering) to help render mesh to have better continuity as shown in Fig. 2.

To correctly simulate the bearing stress in the bolt hole, the model also includes a 1.6 mm- gap between the bolt and the hole since standard bolt holes are 1.6 mm larger than the bolt diameter. Because complete contact between the bolt and the plate is not possible, the ABAQUS contact element, GAP, was used. Contact technique (as shown in Fig. 3) used in these models was slightly different from that employed previously by Ashakul (2004). HARD contact, along with FINITE SLIDING options, was used instead of SMALL SLIDING in the zone where there was no initial contact. For the zone with initial contact, TIE constraint was employed. Friction between the bolt and its hole was neglected. Stress-strain relationship employed was elastic perfectly plastic, with bolt tensile rupture strength of 607 MPa to account for threads in shear plane. However, this number excluded the bolt group effect, which would reduce the strength for further 20 percent. Due to a lack of real stress-strain relationship, an elastic perfectly-plastic strain relationship was

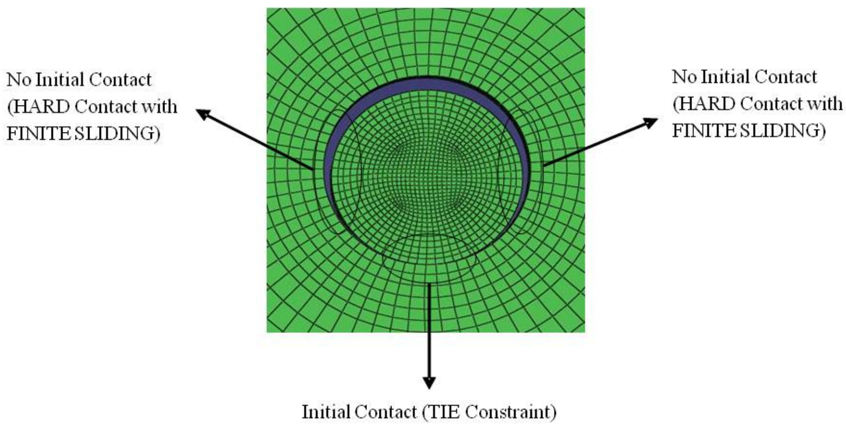


Fig. 3 Contact options used in models

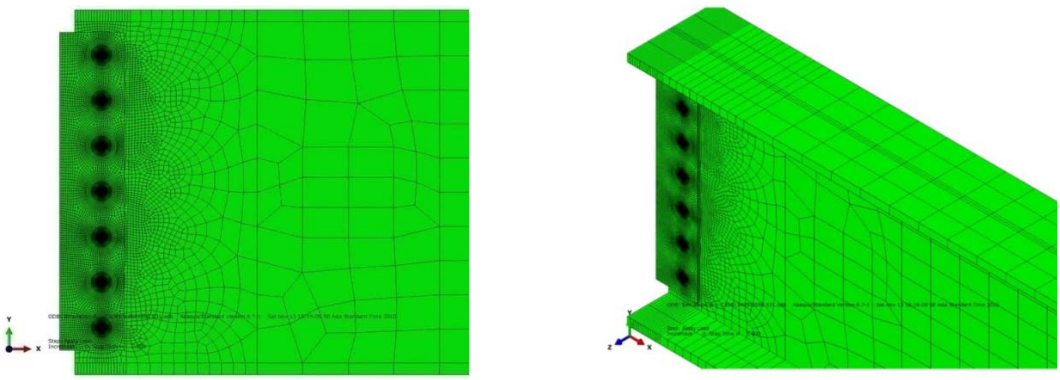


Fig. 4 Complete finite element model

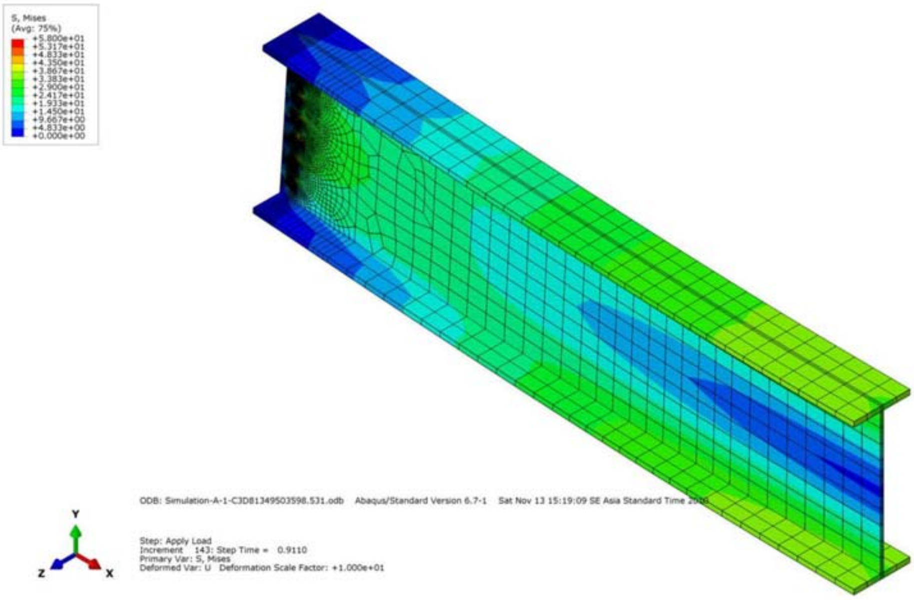


Fig. 5 Beam stress and deformation in model

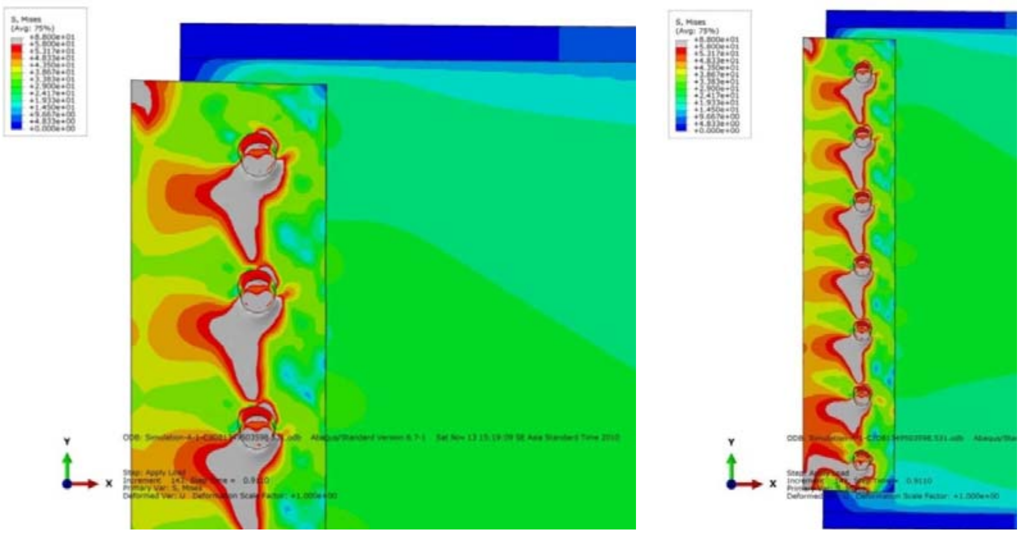


Fig. 6 Magnified deformed shape of plate and its stress in the model

assumed for the E70 welds ($F_y = 483$ MPa) and high strength bolts, and an elastic perfectly-plastic strain relationship with an additional strain-hardening portion was used for the plate and beam material. The stress-strain relationship was converted into true stress and strain as required for ABAQUS input. The Poisson ratio used was 0.3 for every steel material. A complete finite element model and a series of stresses and deformation occurred in the model are illustrated in Figs. 4 through 7.

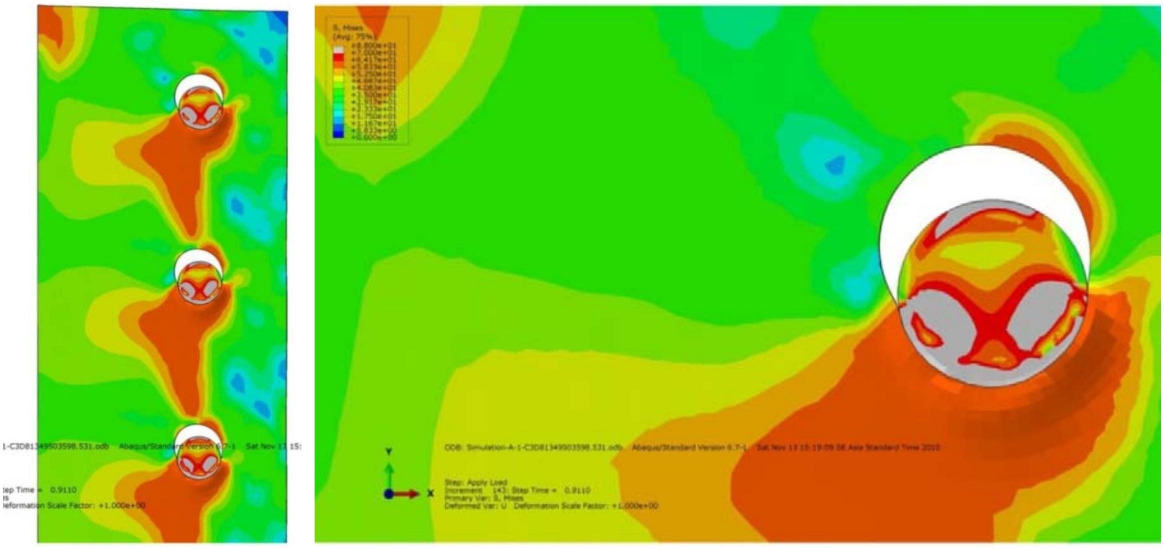


Fig. 7 Magnified deformed shapes of bolt and its hole in plate

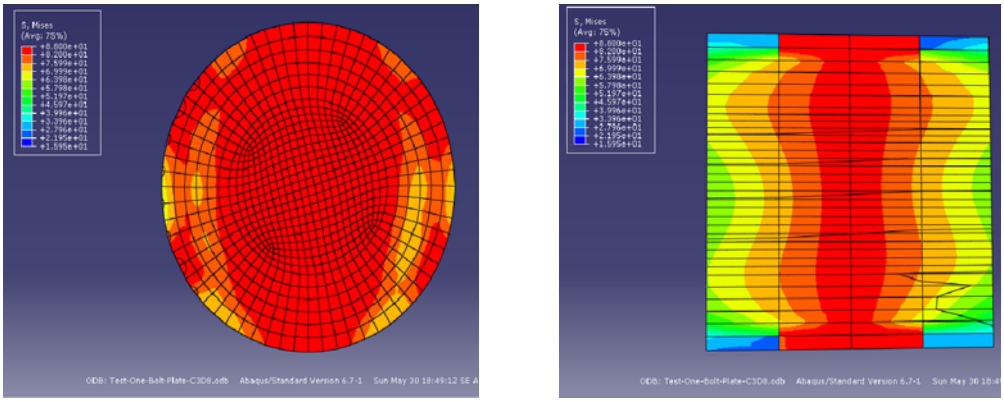


Fig. 8 Stress in bolt when failure judged to have occurred

The model was judged to have failed when stress in any bolt reached its shear rupture. This could be clarified by monitoring the resultant force acting on the bolt, or the von Mises stress in the bolt. Illustration of the bolt when failure occurred is shown in Fig. 8.

2.2 Model verification

The first step, which was to verify the validity and accuracy of the model, was done by creating models to simulate the results from full-scale tests carried out by Astaneh (1989) and Metzger (2006) and a finite element model created by Ashakul (2004). The 7-A325N bolt connection from Astaneh's research, the 5- and 7-A325N bolt connections with Gr.50 plate from Metzger's research, and the 7-bolt finite element model from Ashakul's research were selected. This finite element model by Ashakul (2004) was included because it was one of the models used in his research to verify the model setup with results from five full-scale tests by Astaneh *et al*. (1988) and four full-scale tests by Sakar (1992). If the result of model Ashakul 7-bolt in this research was in good agreement with that of Ashakul's 7-bolt model, it could be concluded that the modeling technique employed would also be able to simulate nine full-scale tests used in Ashakul's research.

Strengths of materials used in the simulations were similar to the numbers from test reports with one exception that bolt shear strength used in Model Ashakul 7-bolt was 364 MPa, not 455 MPa used by Ashakul. This 20 percent difference was due to thread condition because Ashakul had use the strength of A325X bolt in his model whereas Model Ashakul 7-bolt used that of A325N bolt to adhere to the bolt type generally used in this research.

Results from finite element models showed good agreement with numbers from full-scale tests with accuracy up to 90 percent as shown in Table 1. It should be noted that the result of Model Ashakul 7-bolt was nearly 20 percent different from that of Ashakul's model, the number equal to difference in bolt shear strength used in the models.

2.3 Details of finite element models in research

To investigate effects of plate properties on shear strength of the bolt group in the connection, 23 finite element models of 7-bolt connections were set up with various plate thicknesses and strengths, with dimensions in accordance with criteria of the conventional configuration (AISC 2010). The setup was primarily aimed for the connection to fail with bolt shear rupture; therefore, related limit states were carefully prevented by selection of proper plate dimensions. Different varied plate details were plate thicknesses from 7.9 mm through 15.9 mm with 1.6 mm increment, and plate yield strength from 248 MPa through 393 MPa with 48 MPa increment for investigation purposes. Bolt tensile strength used was 607 MPa, which was the full strength of an A325N bolt without deduction of the bolt group effect. Bolt diameters used were 19.0 mm and 22.2 mm so that use of the plate thickness/bolt diameter ratio (t_p/d_b) could also be verified. In addition, all the beams used in the models were Gr.50 ($F_y = 345$ MPa) beams to exclude any effects that beams might have on bolt shear strength of the connections. All details of the finite element models carried out in the research are demonstrated in Table 2. Two cases of connections under direct shear condition, which strength of the connection would not be disturbed by rotational dem and, were set up so that any strength reduction in other simulations could be clearly monitored.

Single plate models with 19.0 mm bolts used 4.88-m W610×125 Gr.50 beam ($F_y = 345$ MPa) while those with 22.2 mm bolts used 4.88-m W610×155 Gr.50 beam. The beam length used in this research was identical to what has been used in the past research in an attempt to construct finite element models of single plate shear connections that would fail with bolt shear rupture while

Table 1 Comparison between test results and simulation predictions

Test	Plate yield strength, MPa	Bolt shear strength, MPa	Test results, kN	Simulation prediction, kN	Ratio of prediction/test
Astaneh 7-bolt	245	N/A	712	694	0.98
Metzger 5-bolt	470	478	649	641	0.99
Metzger 7-bolt	470	423	770	698	0.91
Ashakul 7-bolt*	245	455	841	694	0.82

Table 2 Details of finite element models

Simulation	Bolt diameter, mm	Plate		
		Dimension, mm (thickness × width × length)	F_y, MPa	F_u, MPa
D-1 (Direct Shear)	19.0	9.50 × 108 × 533	248	414
S-1	19.0	9.50 × 108 × 533	245	421
S-2	19.0	11.1 × 108 × 533	248	414
S-3	19.0	12.7 × 108 × 533	248	414
S-4	19.0	14.3 × 108 × 533	248	414
S-5	19.0	7.90 × 108 × 533	296	465
S-6	19.0	9.50 × 108 × 533	296	465
S-7	19.0	11.1 × 108 × 533	296	465
S-8	19.0	12.7 × 108 × 533	296	465
S-9	19.0	7.90 × 108 × 533	345	517
S-10	19.0	9.50 × 108 × 533	345	517
S-11	19.0	11.1 × 108 × 533	345	517
S-12	19.0	7.90 × 108 × 533	393	569
S-13	19.0	9.50 × 108 × 533	393	569
D-2 (Direct Shear)	22.2	11.1 × 121 × 546	248	414
S-14	22.2	11.1 × 121 × 546	248	414
S-15	22.2	12.7 × 121 × 546	248	414
S-16	22.2	14.3 × 121 × 546	248	414
S-17	22.2	15.9 × 121 × 546	248	414
S-18	22.2	11.1 × 121 × 546	296	465
S-19	22.2	11.1 × 121 × 546	345	517
S-20	22.2	11.1 × 121 × 546	393	569
S-21	22.2	15.9 × 121 × 546	393	569

the beam would also reach M_p simultaneously. This proved difficult to achieve as bolt shear rupture generally was judged to have occurred when rotation was approximately 0.01 Radian where the beam behavior remained elastic. Since the focus of this research was to study the effect of plate materials on shear strength of the bolt group, and it was previously observed that the beam rotation in all the models was primarily the function of the beam size and length (Ashakul 2004), rotational behavior of both the beam and the connection itself was not pursued. Further study beam rotation may be necessary to clarify this behavioral aspect.

AISC Design Manual (2010) suggests that plate thickness used should be in accordance with Inequalities (3) and (4). Therefore, plate thickness used in association with 19.0 mm bolts should not be greater than 7.90 mm (0.417 terms of t_p/d_b ratio), and plate thickness used with 22.2 mm bolts should not be greater than 12.7 mm (0.429 in terms of t_p/d_b ratio.) However, many finite element models were deliberately created with plate thicknesses greater than limitation to investigate the behavior of the connection affected by using excessive plate thickness, and any correlation, if exists, between plate yield strength and thickness in affecting bolt shear strength of the connection.

3. Results and discussion

For discussion purposes, results from 23 finite element models were separated into two groups: 19.0-mm bolt and 22.2-mm bolt groups in tables arranged to demonstrate effects on plate properties on shear strength of the bolt group in the connection. Names and results of models are demonstrated in Tables 3 through 6.

Table 3 presents numbers of finite element models of 19.0 mm-bolt connections with various t_p/d_b ratios and plate yield strengths while Table 4 shows the results, which was shear strength of a bolt group, of the corresponding models. Tables 5 and 6 are similar to Tables 3 and 4 except that bolts in the models are all 22.2- mm bolts.

Table 3 Finite element models with 19.0-mm bolt with various plate properties

t_p/d_b ratio	Model				
	Direct shear	F_y, MPa			
		348	296	345	393
0.417*	-	-	S-5	S-9	S-12
0.500	D-1	S-1	S-6	S-10	S-13
0.583	-	S-2	S-7	S-11	-
0.667	-	S-3	S-8	-	-
0.750	-	S-4	-	-	-

Note: * Ratio in accordance with AISC newly recommended values

Table 4 Shear strength of bolt group of finite element models shown in Table 3

t_p/d_b ratio	Shear strength of bolt group, kN (Ratio of shear strength of bolt group/nominal shear)				
	Direct shear	F_y, MPa			
		248	296	345	393
0.417*	-	-	701 (0.96)	665 (0.92)	646 (0.89)
0.500	714	694 (0.95)	638 (0.88)	623 (0.86)	618 (0.85)
0.583	-	629 (0.87)	621 (0.85)	615 (0.85)	-
0.667	-	621 (0.85)	617 (0.85)	-	-
0.750	-	613 (0.84)	-	-	-

Note: * Ratio in accordance with AISC newly recommended values
Nominal bolt shear strength = 7 × 104 = 726 kN

Table 5 Finite element models with 22.2-mm bolt with various plate properties

t_p/d_b ratio	Model				
	Direct shear	F_y, MPa			
		248	296	345	393
0.500	D -2	S-14	S-18	S-19	S-20
0.571	-	S-15	-	-	-
0.643	-	S-16	-	-	-
0.714	-	S-17	-	-	S-21

Note: * Ratio in accordance with AISC newly recommended values

Table 6 Shear strength of bolt group of finite element models shown in Table 5

t_p/d_b ratio	Shear strength of bolt group, kN (Ratio of shear strength of bolt group/nominal shear)				
	Direct shear	F_y, MPa			
		248	296	345	393
0.500	959	948 (0.96)	891 (0.89)	870 (0.88)	857 (0.87)
0.571	-	897 (0.91)	-	-	-
0.643	-	879 (0.89)	-	-	-
0.714	-	857 (0.87)	-	-	850 (0.86)

Note: * Ratio in accordance with AISC newly recommended values.
Nominal bolt shear strength = 7*141 = 989 kN

Forces acting on bolts, presented in terms of the horizontal and vertical force components, and the resultants (vector sum), were summarized in Tables 7 and 8.

3.1 Effects of plate thickness on shear strength of bolt group

Results of Simulations with 19.0-mm Bolts

From Table 4 which demonstrates shear strength of a bolt group in connections with 19.0-mm bolts, when plate thickness was increased in models with plate yield strength equal to 248 MPa so that t_p/d_b ratios became 0.500, 0.583, 0.667 and 0.750, shear strength of a bolt group in the connections decreased by 5, 13, 15, and 16 percent compared to its nominal shear strength, respectively.

For models with plate yield strength equal to 296 MPa, when plate thickness was increased so that t_p/d_b ratios became 0.417, 0.500, 0.583 and 0.667, shear strength of a bolt group in the connections decreased by 5, 12, and 15 percent (for the last two models) compared to its nominal shear strength, respectively.

For models with plate yield strength equal to 345 MPa, shear strength of a bolt group the connections with t_p/d_b ratios of 0.417, 0.500, and 0.583 decreased by 8, 14, and 15 percent compared to its nominal shear strength, respectively.

A plot demonstrating changes of shear strength of the bolt group in 19.0-mm bolt connections with respect to different t_p/d_b ratios is shown in Fig. 9.

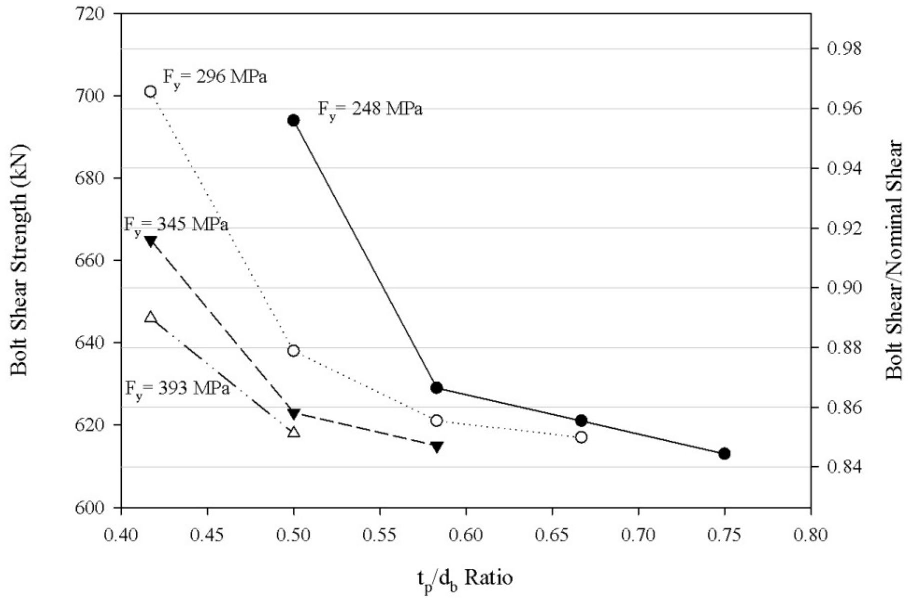


Fig. 9 Shear strength of in 19.0-mm bolt group in connection vs. t_p/d_b ratios

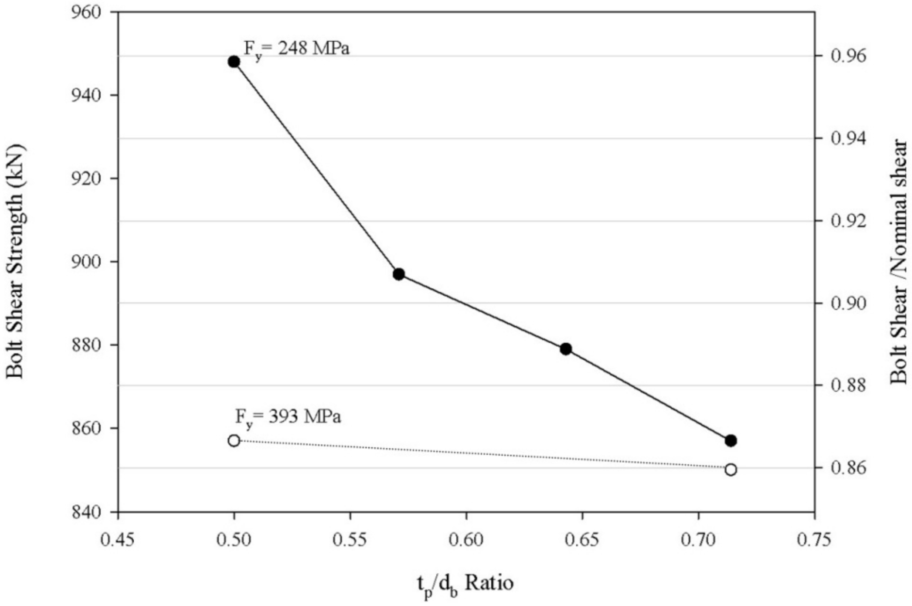


Fig. 10 Shear strength of 22.2-mm bolt group in connections vs. t_p/d_b ratios

Results of simulations with 22.2-mm bolts

From Table 6 which shows shear strength of a bolt group in connections with 22.2-mm bolts, when t_p/d_b ratios in models with plate yield strength equal to 248 MPa were 0.500, 0.571, 0.643 and 0.714, shear strength of the bolt group decreased by 4, 9, 11, and 13 percent compared to its nominal shear strength, respectively.

For models with plate yield strength equal to 393 MPa, shear strength of the bolt group in connections with t_p/d_b ratios of 0.500 and 0.714 decreased by 13 and 14 percent compared to its nominal shear strength, respectively.

A plot illustrating changes of shear strength of the bolt group in 19.0-mm bolt connections with respect to different t_p/d_b ratios is shown in Fig. 10.

From finite element results, including illustrations in Figs. 7 and 8, it can be seen that increase of plate thickness, which caused the t_p/d_b ratio to increase, affected shear strength of the bolt group in the connection. The effect was rapid once the ratio used exceeded 0.500 and then started to converge to approximately 15 percent reduction when the ratio was much greater.

3.2 Effects of plate yield strength on shear strength of bolt group

Results of simulations with 19.0-mm bolts

From results of 19.0-mm bolt connections shown in Table 4, for models with plate thicknesses equal to 7.90 mm and the t_p/d_b ratio equal to 0.417, shear strength of a bolt group in simulations with plate yield strength of 296, 345, and 393 MPa decreased by 4, 8, and 11 percent compared to its nominal shear strength, respectively.

For models with plate thicknesses equal to 9.50 mm and the t_p/d_b ratio equal to 0.500, shear strength of a bolt group in simulations with plate yield strength of 248, 296, 345, and 393 MPa decreased by 5, 12, 14, and 15 percent compared to its nominal shear strength, respectively.

For models with plate thicknesses equal to 11.1 mm and the t_p/d_b ratio equal to 0.583, shear strength of a bolt group in simulations with plate yield strength of 248, 296, and 345 MPa decreased by 13 and 15 percent for the last two models compared to its nominal shear strength, respectively.

For models with plate thicknesses equal to 12.7 mm and the t_p/d_b ratio equal to 0.667, shear strength of a bolt group in simulations with plate yield strength of 248 and 296 MPa decreased by 15 percent compared to its nominal shear strength. Decrease in shear strength became constant in these simulations, which had a high t_p/d_b ratio.

A plot illustrating changes of shear strength of the bolt group in 19.0-mm bolt connections with respect to different plate yield strength is shown in Fig. 11.

Results of Simulations with 22.2-mm Bolts

From results of 22.2-mm bolt connections in Table 6, for models with plate thicknesses equal to 9.50 mm and the t_p/d_b ratio equal to 0.500, shear strength of a bolt group in simulations with plate yield strength of 248, 296, 345, and 393 MPa decreased by 5, 12, 14, and 15 percent compared to its nominal shear strength, respectively. Decrease of shear strength of the bolt group in these 22.2-mm bolt connections was almost identical to that in 19.0-mm bolt connections.

For models with plate thicknesses equal to 15.9 mm and the t_p/d_b ratio equal to 0.714, shear strength of a bolt group in simulations with plate yield strength of 248 and 393 MPa decreased by 15 percent compared to its nominal shear strength, similarly to results of 19.0-mm bolt simulations

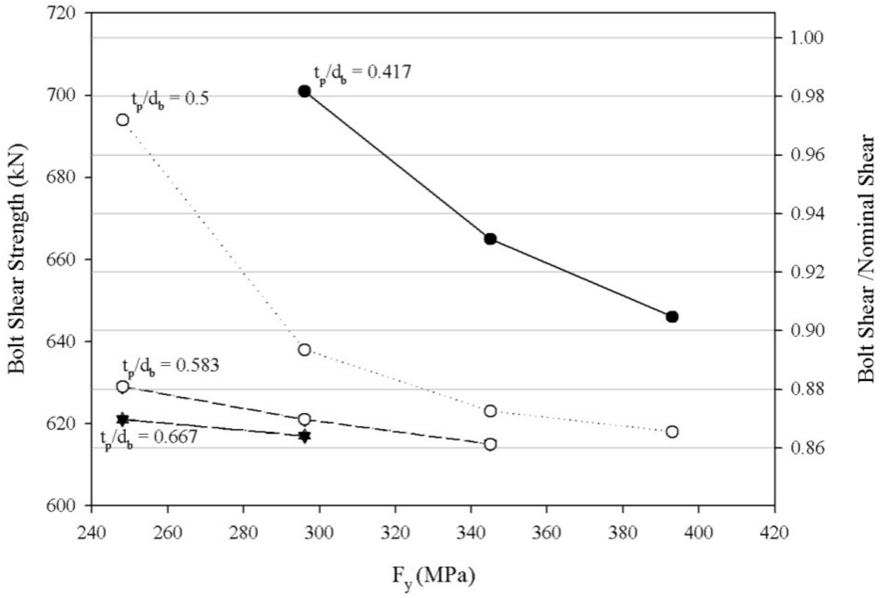


Fig. 11 Shear strength of 19.0-mm bolt group in connections vs. plate yield strength

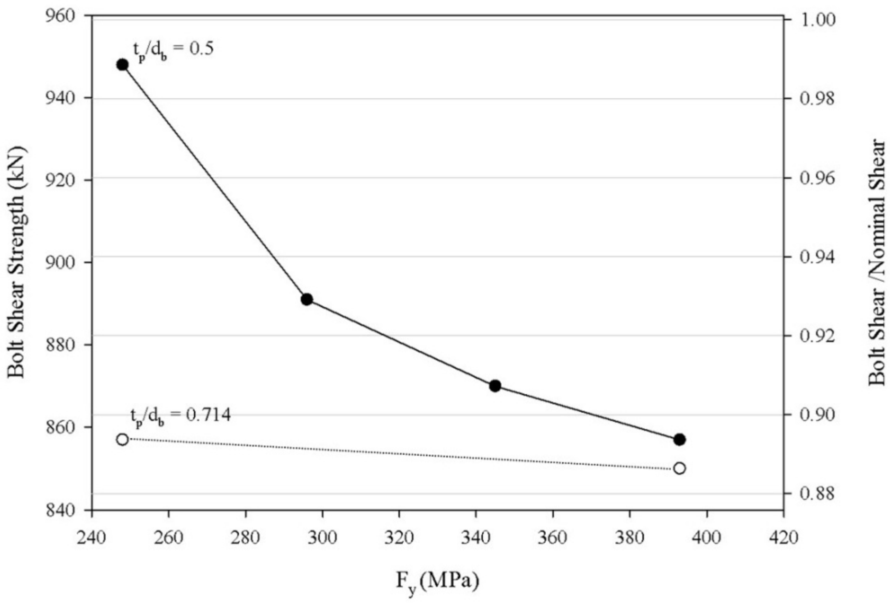


Fig. 12 Shear strength of 22.2-mm bolt group in connections vs. plate yield strength

with had a high t_p/d_b ratio.

A plot demonstrating changes of shear strength of the bolt group in 22.2-mm bolt connections with respect to different plate yield strength is shown in Fig. 12.

From the results, it can be seen that increase of plate yield strength had similar effects as increase of t_p/d_b ratio. The impact was significant once plate yield strength used exceeded 248 MPa and started to converge to approximately 15 percent reduction of bolt shear strength when plate yield strength was much greater than 248 MPa.

3.3 Correlation between plate thickness and yield strength

From discussions in Topics 3.1 and 3.2, plate yield strength had a greater impact on shear strength of the bolt group in the connection when t_p/d_b ratios used were not greater than 0.500. Once the ratios used exceeded 0.500, bolt shear strength was reduced up to 15 percent regardless of plate yield strength.

Using t_p/d_b ratio beyond the value of 0.500 with plate yield strength of 248 MPa also had a great impact on the bolt shear strength in similar fashion to using plate yield strength beyond 248 MPa with the t_p/d_b ratio not greater than 0.500. Use of plate yield strength greater than 248 MPa when the t_p/d_b ratio already exceeded 0.500 did not have a significant impact to shear strength of the bolt group that was already reduced.

It should be noted that shear strength of the bolt group in simulations with t_p/d_b ratios lower than 0.500 was also affected by plate yield strength, but not as much as simulations with high t_p/d_b ratios.

3.4 Validity of t_p/d_b ratio

Validity of using a t_p/d_b ratio to control selection of a plate thickness was investigated by changing of a bolt diameter used from 19.0 mm to 22.2 mm while maintaining the t_p/d_b ratio. This was done because most, if not all, of the full-scale tests carried out in the past by many researchers were done with 19.0 mm bolts.

Results from finite element analyses indicated that the t_p/d_b ratio could be used effectively to select a plate thickness in conjunction with a bolt diameter in designing single plate shear connections. However, the ratio should be limited to 0.500, along with use of an A36 plate (F_y = 248 MPa).

3.5 Investigation of forces acting on bolts

Because bolt movement that caused bolt plowing was always in a diagonal direction; therefore, forces acting on bolts against the movement could be viewed as horizontal and vertical force components as shown in Fig. 13. Forces acting on bolts were summarized in Tables 7 and 8. Vertical force components were always upward, whereas horizontal force components could be in the direction towards the beam or towards the support (as indicated by the minus sign).

From Tables 7 and 8, the sum of vertical forces acting on each bolt was equal to shear strength of the bolt group of the connection. The magnitude of the vertical forces in one bolt group was almost equal, with the greatest difference from the average values being no more than 6.5 percent (5.52 kN) in one case for 19.0-mm bolt connections, and 5.5 percent (6.41 kN) for 22.2-mm bolt connections.

The sum of horizontal forces indicated that there was a compressive force in the plate directing towards the support. The presence of this horizontal compressive force in the plate, even though generally small (except for Model S-14), suggests that it might be necessary to check plate buckling strength of the connection, especially in extended configuration with large a-distance (Muir and Hewitt 2009).

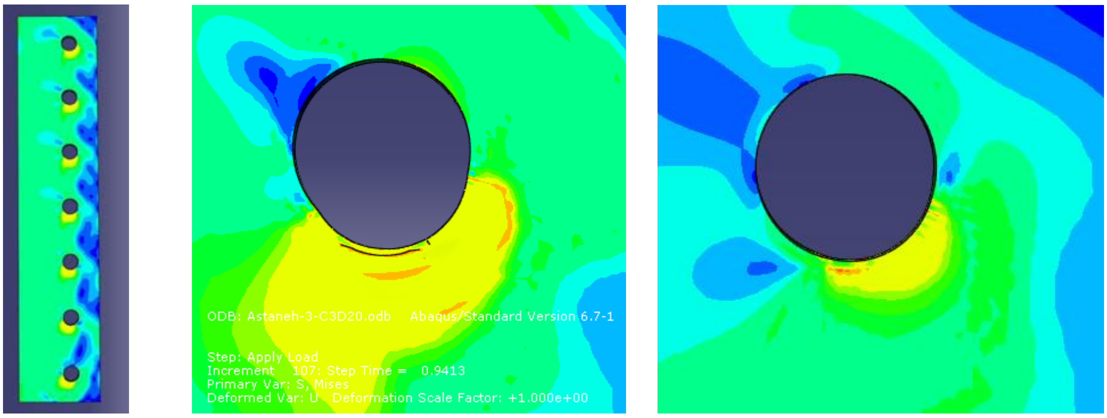


Fig. 13 Bolt movement in plate inducing horizontal and vertical forces acting on bolts

Table 7 Forces acting on bolts in 19.0-mm bolt connections

Bolt	**Plate yield strength = 248 MPa**			
	S-1 $t_p/d_b = 0.500$	S-2 $t_p/d_b = 0.583$	S-3 $t_p/d_b = 0.667$	S-4 $t_p/d_b = 0.750$
		Vertical force (kN)		
1	99.8	92.2	89.5	90.5
2	101	92.2	93.2	89.4
3	101	90.0	87.9	85.8
4	100	88.3	85.4	84.1
5	98.5	89.0	88.9	85.7
6	97.2	90.0	91.3	89.4
7	95.5	86.7	84.2	88.6
Total	**694**	**628**	**620**	**613**
Average	**99.1**	**89.8**	**88.6**	**87.6**
Deviation (%)	**3.56 (3.59)**	**3.07 (3.42)**	**4.63 (5.22)**	**3.56 (4.06)**
		Horizontal Force (kN)		
1	28.6	47.6	52.1	50.8
2	21.7	29.7	30.2	32.8
3	9.61	12.7	13.1	14.1
4	-3.02	-2.40	-2.18	-2.14
5	-14.7	-17.0	-16.9	-18.5
6	-24.6	-34.1	-35.1	-35.9
7	-27.4	-49.7	-55.6	-53.1
Total	**-9.83**	**-13.3**	**-14.4**	**-11.9**

Table 7 Continued

Bolt	Plate yield strength = 248 MPa			
	S-1 t_p/d_b = 0.500	S-2 t_p/d_b = 0.583	S-3 t_p/d_b = 0.667	S-4 t_p/d_b = 0.750
Resultants (kN)				
1	104	104	104	104
2	103	96.9	98.0	95.2
3	102	90.9	88.8	86.9
4	100	88.4	85.5	84.1
5	99.6	90.6	90.5	87.7
6	100	96.3	97.8	96.3
7	99.4	99.9	101	103

Table 7 Continued

Bolt	Plate yield strength = 296 MPa			
	S-5 t_p/d_b = 0.417	S-6 t_p/d_b = 0.500	S-7 t_p/d_b = 0.583	S-8 t_p/d_b = 0.667
Vertical force (kN)				
1	101	92.3	89.9	87.2
2	101	93.3	93.2	93.6
3	102	92.5	87.6	88.8
4	101	91.2	85.6	85.0
5	99.6	91.4	88.8	90.4
6	98.8	89.7	91.2	88.1
7	96.3	87.7	84.6	83.3
Total	**700**	**638**	**621**	**616**
Average	**100**	**91.1**	**88.7**	**88.1**
Deviation (%)	**3.69 (3.69)**	**3.43 (3.76)**	**4.49 (5.07)**	**5.52 (6.26)**
Horizontal force (kN)				
1	22.5	47.5	51.7	56.0
2	16.8	29.9	30.2	31.8
3	7.56	12.9	13.2	12.9
4	-3.02	-2.54	-2.27	-2.18
5	-13.0	-17.3	-17.1	-16.7
6	-20.8	-34.8	-35.2	-31.9
7	-21.6	-49.2	-55.5	-59.6
Total	**-11.6**	**-13.5**	**-15.1**	**-9.61**

Table 7 Continued

Bolt	Plate yield strength = 296 MPa			
	S-5 $t_p/d_b = 0.417$	S-6 $t_p/d_b = 0.500$	S-7 $t_p/d_b = 0.583$	S-8 $t_p/d_b = 0.667$
Resultants (kN)				
1	104	104	104	104
2	103	97.9	97.9	98.9
3	102	93.4	88.6	89.8
4	101	91.2	85.6	85.1
5	100	93.0	90.5	91.9
6	101	96.3	97.8	93.7
7	98.8	101	101	102

Table 7 Continued

Bolt	Plate yield strength				
	345 MPa			393 MPa	
	S-9 $t_p/d_b = 0.417$	S-10 $t_p/d_b = 0.500$	S-11 $t_p/d_b = 0.583$	S-12 $t_p/d_b = 0.417$	S-13 $t_p/d_b = 0.500$
Vertical force (kN)					
1	94.7	90.5	87.5	92.3	88.7
2	97.5	92.4	92.9	95.0	93.0
3	97.2	88.4	87.9	93.5	87.7
4	97.3	87.0	84.5	92.6	85.4
5	95.4	88.8	87.5	93.1	89.1
6	93.6	90.5	91.0	91.2	90.8
7	89.5	85.1	83.2	88.3	83.8
Total	**665**	**623**	**615**	**646**	**618**
Average	**95.1**	**89.0**	**87.8**	**92.3**	**88.3**
Deviation (%)	**5.52 (5.80)**	**3.87 (4.35)**	**5.12 (5.83)**	**3.96 (4.29)**	**4.63 (5.24)**
Horizontal force (kN)					
1	42.7	50.0	54.7	47.1	53.0
2	26.3	30.1	31.1	29.0	30.7
3	12.1	13.3	12.7	12.5	13.0
4	-3.69	-2.36	-2.22	-3.69	-2.27
5	-17.1	-17.7	-16.2	-18.4	-17.3
6	-31.6	-35.3	-36.4	-35.5	-35.9
7	-45.0	-53.8	-58.5	-50.9	-57.0
Total	**-16.4**	**-15.9**	**-14.9**	**-20.0**	**-15.7**

Table 7 Continued

Bolt	Plate yield strength				
	345 MPa			393 MPa	
	S-9 t_p/d_b = 0.417	S-10 t_p/d_b = 0.500	S-11 t_p/d_b = 0.583	S-12 t_p/d_b = 0.417	S-13 t_p/d_b = 0.500
	Resultant (kN)				
1	104	103	103	104	103
2	101	97.2	97.9	99.3	97.9
3	98.0	89.4	88.9	94.3	88.6
4	97.4	87.0	84.5	92.6	85.4
5	96.9	90.6	89.0	94.9	90.8
6	98.8	97.2	98.0	97.9	97.6
7	100	101	102	102	101

Note: Bolt nominal strength = 104 kN

Vector sum of forces acting on the bolt that failed in each simulation was equal to nominal bolt shear strength underlining the fact that bolt shear rupture could be judged to have occurred. The resultant force was in agreement with von Mises stress in the bolt (Khampa 2010). The bolts that failed in the simulations were the outermost ones. The magnitude of horizontal forces acting on these bolts was largest compared to others in the group because the outermost bolts underwent the most "bolt plowing" action caused by beam rotation. Since the magnitude of the vertical force acting on each bolt in the connection was not substantially different, it was clear that the presence of the horizontal forces was the reason that caused bolt failure and prevent the bolt from carrying more vertical shear force from the beam.

Table 8 Forces acting on bolts in 22.2-mm bolt connections

Bolt	Plate yield strength = 248 MPa			
	S-14 t_p/d = 0.500	S-15 t_p/d = 0.571	S-16 t_p/d = 0.643	S-17 t_p/d = 0.714
	Vertical forces (kN)			
1	138	132	128	127
2	138	131	128	126
3	138	131	125	118
4	136	128	124	116
5	134	125	123	121
6	133	125	125	127
7	132	124	125	122
Total	**948**	**897**	**879**	**857**
Average	**135**	**128**	**126**	**122**
Deviation (%)	**3.43 (2.53)**	**4.31 (3.37)**	**2.49 (1.98)**	**6.41 (5.23)**

Table 8 Continued

Bolt	Plate yield strength = 248 MPa			
	S-14 t_p/d = 0.500	S-15 t_p/d = 0.571	S-16 t_p/d = 0.643	S-17 t_p/d = 0.714
Horizontal forces (kN)				
1	31.1	48.9	59.7	60.5
2	20.2	30.4	36.4	34.5
3	4.27	10.1	14.8	14.9
4	-10.8	-7.07	-5.60	-4.27
5	-23.9	22.9	-25.4	-21.7
6	-32.4	-36.4	-43.4	-42.7
7	-33.6	-44.8	-61.4	-64.5
Total	**-45.1**	**-21.8**	**-24.8**	**-23.2**
Resultants (kN)				
1	141	141	141	140
2	139	135	133	131
3	138	131	126	119
4	136	128	124	116
5	136	127	126	123
6	137	130	133	133
7	136	132	139	138

Table 8 Continued

Bolt	Plate yield strength			
	296 MPa	**345 MPa**	**393 MPa**	
	S-18 t_p/d = 0.500	S-19 t_p/d = 0.500	S-20 t_p/d = 0.500	S-21 t_p/d = 0.714
Vertical shear (kN)				
1	131	129	127	121
2	128	126	126	125
3	129	123	119	120
4	127	122	118	117
5	125	123	121	125
6	125	125	126	126
7	119	123	120	117
Total	**891**	**870**	**857**	**852**
Average	**127**	**124**	**122**	**122**
Deviation (%)	**8.45 (6.64)**	**4.31 (3.47)**	**4.63 (3.78)**	**4.72 (3.87)**

Table 8 Continued

Bolt	Plate yield strength			
	296 MPa	345 MPa	393 MPa	
	S-18 $t_p/d = 0.500$	S-19 $t_p/d = 0.500$	S-20 $t_p/d = 0.500$	S-21 $t_p/d = 0.714$
Horizontal shear (kN)				
1	52.8	57.9	60.0	70.5
2	30.1	35.2	35.2	39.5
3	11.5	14.8	15.3	13.9
4	7.30	5.29	-4.76	-3.29
5	-22.4	-23.1	-23.3	-20.3
6	-39.4	-44.0	-44.1	-48.0
7	-54.4	-62.6	-67.7	-77.1
Total	**-29.0**	**-16.5**	**-29.2**	**-24.7**
Resultants (kN)				
1	141	141	141	140
2	131	131	131	131
3	130	124	120	121
4	127	122	118	117
5	127	125	123	127
6	131	132	133	135
7	131	138	138	140

Note: Bolt nominal strength = 141 kN

In the same manner as Tables 4 and 6, Tables 9 and 10 are arranged to demonstrate the effect of plate materials on the magnitude of horizontal forces acting on bolts. It can be seen from Table 9 that the magnitude of the horizontal forces increased when plate yield strength increased. Average horizontal forces acting on outermost bolts in 19.0-mm bolt for connections with t_p/d_b ratio equal to 0.417 were 22.1, 43.9, and 49.0 kN when plate yield strengths were equal to 296, 345, and 393 MPa, respectively; and for those with a t_p/d_b ratio equal to 0.500 were 28.0, 48.4, 51.9, and 55.0 kN when plate yield strengths were equal to 248, 296, 345, and 393 MPa, respectively. Average horizontal forces on outermost bolts for connections with t_p/d_b ratio of 0.583, 0.667, and 0.75, which exceed the recommended number by AISC, were greater than 44.5 Kn.

Table 10, which presents the horizontal forces acting on bolts in 22.2-mm bolt connections, contains information with the same trend as that in Table 9, i.e., the magnitude of these forces are significant in connections with high plate yield strength and t_p/d_b ratios.

It can be clearly seen from Tables 9 and 10 that the magnitude of horizontal forces became significant when plate yield strength became greater than 248 MPa. The magnitude of these forces was also substantial when t_p/d_b ratio exceeded 0.500. The magnitude was up to 50% of bolt shear strength when plate yield strength was 345 MPa or when the t_p/d_b ratios was greater than 0.500. This could decrease the magnitude of the vertical force acting on the bolt and, ultimately, shear

Table 9 Average horizontal forces on outermost bolts in 19.0-mm bolt connections

t_p/d_b ratio	Horizontal force on bolts (kN) (Percentage of horizontal force/nominal shear)			
	F_y (MPa)			
	248	296	345	393
0.417	-	22.0 (21.2)	43.8 (42.2)	49.0 (47.2)
0.500	28.0 (26.9)	48.4 (46.6)	51.9 (50.0)	55.0 (53.0)
0.583	48.7 (46.9)	53.6 (51.6)	56.6 (54.5)	-
0.667	53.9 (51.9)	57.7 (55.7)	-	-
0.750	51.9 (50.0)	-	-	-

Table 10 Average horizontal forces on outermost bolts in 22.2-mm bolt connections

t_p/d_b ratio	Horizontal force on bolts (kN) (Percentage of horizontal force/nominal shear)			
	F_y (MPa)			
	248	296	345	393
0.500	32.3 (22.9)	53.6 (38.0)	60.2 (42.7)	63.9 (45.2)
0.571	46.9 (33.2)	-	-	-
0.643	60.6 (42.9)	-	-	-
0.714	62.5 (44.2)	-	-	73.9 (52.3)

Table 11 Maximum plate thicknesses for conventional single plate connections (AISC 2010)

$n \le 5$			$n > 5$		
d_b (mm)	Maximum t_p (mm)	t_p/d_b ratios	d_b (mm)	Maximum t_p (mm)	t_p/d_b ratios
19.0	11.1	0.583	19.0	7.90	0.417
22.2	12.7	0.571	22.2	9.50	0.429
25.4	14.3	0.562	25.4	11.1	0.438

Table 12 Design shear strength of bolt group under eccentricities

Maximum e	e_{max} = 44.4 mm				e_{max} = 88.9 mm						
No.of bolts	2	3	4	5	6	7	8	9	10	11	12
% of design shear strength	0.64	0.78	0.86	0.90	0.79	0.83	0.86	0.88	0.90	0.91	0.92

strength of the bolt group, up to 15%. The effect of plate yield strength being greater than 248 MPa on the magnitude of the horizontal forces acting on bolts, and collectively shear strength of the bolt group, is similar to the that of t_p/d_b ratio being greater than 0.500. Magnitude of horizontal forces reflects the reduction of shear strength of the bolt group previously discussed in Topics 3.1 and 3.2.

4. Conclusions

Results from finite element models suggest that shear strength of a bolt group was reduced once plate yield strength was increased. Reduction was rapid when plate yield strength became greater than 248 MPa with reduction up to 15 percent. Likewise, shear strength of the bolt group in a connection was reduced up to 10 percent immediately when the t_p/d_b ratio used exceeded the 0.500 limit. Results also show that when plate yield strength was greater than 248 MPa, but the t_p/d_b ratio was lower than 0.500, shear strength of the bolt group in the connection was not reduced as much as when the ratio was 0.500.

It is evident that use of plate with yield strength greater than 248 MPa or use of plate thickness that results in the t_p/d_b ratio exceeds 0.500 will similarly affect shear strength of the bolt group in the connection.

Design strength values available in the previous Design Manual (AISC 2005) for conventional single plate shear connections with t_p/d_b ratio greater than 0.500 were removed from the current one (AISC 2010) and replaced by suggestions regarding the plate thicknesses and eccentricities for two connection categories: connections with number of bolts not greater than five; and ones with number of bolts greater than five.

Maximum plate thicknesses that can be used in conjunction with the popularly used bolt sizes are displayed in Table 11. It can be seen that when the number of bolts is not greater than five, the resulting t_p/d_b ratio is greater than 0.500 due to the addition of the 1.6 mm term in Equation (3), where as the resulting t_p/d_b ratio for connections with the number of bolts greater than five is less than 0.500 due to the subtraction of the 1.6 mm term in Equation (4). As a result, the maximum plate thickness as suggested by the current Design Manual agrees with results from finite element results. Even though the maximum plate thickness for connections with the number of bolts not greater than five is greater than 0.500, these small connections tend to have smaller magnitude of horizontal forces acting on bolts (Ashakul 2004) and may not have as much reduction of shear strength of bolt group as multiple-bolt connections. Nevertheless, one should still be cautious when using the t_p/d_b ratio greater than 0.500.

In terms of eccentricity, current Design Manual (2010) suggests that, for conventional configuration, e = a/2 be used for connections with the number of bolts not greater than five, whereas e = a be used for connections with the number of bolts greater than five. This is translated to maximum eccentricities being no more than 44.4 mm for up to 5-bolt connections and 88.9 mm for connections with six bolts or more. Percentage of design shear strength of bolt groups compared to concentrically loaded bolt group based on theses eccentricities is shown in Table 12. Without considering the effect of plate properties, shear strength of bolt group shall be reduced to as small as 64 percent for a 2-bolt connection and 79 percent for a 6-bolt connection. The shear strength of the bolt group increases once the number of bolts in their respective eccentricity increases. Even though the purpose of finite element models created in this research was not to investigate the relationship between the number of bolts and design shear strength of the bolt group, the evidence of the horizontal force from this research and Ashakul's (2004) suggests that design shear strength may not increase along with the number of bolts. In addition, because plate yield strength in construction is generally higher than the nominal required strength, i.e., actual plate yield strength of A36 steel can be significantly greater than 248 MPa as there is no upper limit for Yield Point in ASTM A36 Specification (2008). As a result, use of connection configurations with a number of bolts equal or greater than six should be done with precautionary bolt strength reduction.

The presence of horizontal forces acting on bolts demonstrates the reason behind the reduction of shear strength of the bolt group. Summation of horizontal forces also indicates that it is possible that plate buckling might occur; therefore, this limit state should be checked, especially when the extended configuration with long a-distance is employed. The magnitude of the summation of tensile force can be as high as 110 kN in 22.2-mm bolt connections. As a result, suggestion that single plate shear connection be designed for tensile force not less than 44.5 kN may be insufficient. An alternative to handling horizontal forces would be an introduction of a slotted hole to the beam web.

It should be noted that this research was carried out by using a 7-bolt configuration only, and that connection configurations with greater number of bolts is likely to have greater strength reduction. In addition, increase of the number of bolt rows in single plate connections has been proven not to reduce ductility of the connection, but shear strength of the bolt group must be further investigated as a t_p/d_b ratio in multiple-row single plate is likely to be greater than 0.500, and behavior of the multiple-row bolt group may not be directly related to that of a single-row bolt group.

From the conclusions, the following suggestions in designing single plate shear connections should be applied along with the current design model:

(1) Bolt shear strength in single plate connections with a number of bolts equal or greater than seven should be reduced by 15 percent to account for strength reduction caused by a plate with high yield strength.
(2) Plate thickness-to-bolt diameter ratio, or t_p/d_b, should be limited to 0.500 to avoid bolt group shear strength reduction.

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