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Shear-Out Capacity of Bolted Connections in Cold-Reduced Steel Sheets

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A/Prof. Lip Teh and Prof. Zhengyi Jiang

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Abstract

Bolted connections are one of the important connection methods and are widely used in civil engineering construction because of their reliability and easy installation. Cold formed steel is also widely used for its high strength, fast forming and accurate dimensions. Therefore, study of the shear-out capacity of cold-rolled steel plate has great significance in providing guidance for engineering practice and improving structural safety and economy. This thesis examines the accuracy of design equations specified in the North American, European and Australian/New Zealand standards for determining the shear-out capacity of bolted connections in cold-reduced steel sheets.

In the study 71 single-bolted specimens were tested with varying end distances, edge distances, sheet thickness, bolt (hole) diameters, steel grades and steel plate cutting methods, in which 60 of them failed by shear-out. The ratios of end distance to bolt diameter ranged from 1.25 to 3.0. Load-displacement graphs were obtained and the deformation and fracture location of the steel plate were recorded, these provided data for the formulation of an equation of the ultimate shear-out capacity of a bolted connection in cold-reduced steel sheet.

Finite element analysis software ABAQUS was used to conduct material and geometric nonlinear analyses including contact and fracture simulations. The thesis shows that the catenary action in a narrow strip downstream of the bolt has a significant effect on the shear-out capacity, and is not accounted for in the design codes. The contribution of catenary action was corroborated through the finite element analysis.

The thesis also shows that the edge distance (as opposed to the end distance) of a bolted connection has insignificant effects on the shear-out capacity, although it affects the failure mode. An additional finding is that shearing (rather than sawing) the end of a cold-reduced steel sheet can sometimes change the failure mode into end splitting, but not to the same extent as for hot-rolled steel plates. Based on the test results of 60 single-bolted specimens having end distances ranging from 1.25 to 3.0 times the bolt diameter, this thesis derives a shear-out equation that is dimensionally consistent, continuous and transparent, and that is much more accurate than the code equations. The proposed equation, which has the bolt diameter as a parameter, is verified against

independent test specimens having end distances as small as 1.0 times the bolt hole diameter.

Keywords: bolted connections, cold-formed steel, shear-out, ultimate load, finite element analysis

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Certification

I, Haoyu Xing, declare that this thesis submitted in fulfilment of the requirements for the conferral of the degree Master of Research Project, from the University of Wollongong, is wholly my own work unless otherwise referenced or acknowledged. This document has not been submitted for qualifications at any other academic institution.

Haoyu Xing

1st June 2019

List of Names or Abbreviations

e	Length of effective shear plane
e_1	Length of gross shear plane
e_n	Length of net shear plane
d_h	Diameter of bolt hole
p_2	Gauge distance
d_b	Diameter of bolt
W	Width of the plate
t	Thickness of the plate
F_y	Yield stress
F_u	Ultimate stress
R_n	Ultimate load predicted by equations
P_t	Test load
c	Coefficient of catenary action
PF	Professional factor of equations
P_m	Mean professional factor

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Chapter 1 Introduction

1.1 Background

Cold-formed steel is widely used in the construction industry since its first introduction in codified standards in 1946. Cold-formed steel members, such as cold-rolled bar stock and sheet, are commonly used in all areas of durable goods manufacture and also in bridges, buildings, storage racks, car bodies, grain bins, highway products, and transmission towers. Cold-formed steel is made by rolling or pressing steel at relatively low temperatures. The material thickness of cold-formed steel members usually ranges from 0.373 mm to 6.35 mm. Countries like the United States, Canada, and Mexico use the North American Specification (AISI) for the design of cold-formed steel members. Member states of the European Union utilise Eurocode 3 for the design of cold-formed structural members. Other nations use various design specifications; many base their building code on AISI. Compared with hot-rolled steel, cold-formed steel has the properties of high strength, low ductility, low lightness and ease of prefabrication and mass production.

Bolted connections are one of the most common elements in construction and machine design. It is one of the important ways of connecting steel structures and is widely used in structural engineering. It is easy to install and can shorten the construction period whilst at the same time it is easy to disassemble. The bearing capacity of the bolted connection is usually taken as the controlling factor in the design of the connection. Therefore, research on the bearing capacity of the connection has great value. There are two main bolt connection designs: tension connections and shear connections. The shear connection, which is mainly discussed in this thesis, transfers the applied load in shear at the bolt shank. It relies on the shear strength of the bolt and the plate.

1.2 Problem Definition

Shear-out failure (also termed tear-out failure), as shown in Figure 1.1, according to Teh and Uz (2015), is distinct from another failure mode termed bearing failure. The specifications in American, European and Australian/New Zealand standards give equations to predict the shear-out ultimate load. However, these equations

differ not only in coefficients but also in variables, and always overestimate or underestimate the shear-out capacity of the bolted connection. Teh and Uz (2015) have shown that the shear-out capacity given by the North American cold-formed steel specification is too conservative for hot-rolled steel bolted connections, but it is uncertain whether the same outcome applies to cold-reduced sheet steel connections as the latter often have reduced material ductility. To the authors' knowledge, the European and Australian shear-out provisions have never been checked against cold-reduced sheet steel connections since they were adopted from the respective provisions for hot-rolled steel connections. There does not appear to be a systematic study on the shear-out capacity of bolted connections in cold-reduced sheet steel, although He and Wang (2011) presented test results of such specimens in four configurations. The ratios of end distance to bolt hole diameter tested by them ranged from 1.0 to 1.5 only. As a result, a high safety coefficient is used when designing a steel construction, causing an excess use of steel and extra cost.

Figure 1.1. Shear-out failure

1.3 Aim

This thesis seeks to get an accurate and consistent equation to predict the ultimate load of bolted connections in cold-reduced steel sheets. This equation, once determined, can be applied in steel construction design, which as a result can make good use of the strength of steel and help make the connection safe.

1.4 Objectives

The following is a list of objectives which will contribute to achieving the aim presented in Section 1.3.

1. Review, analyse and critically evaluate existing specifications and literature regarding the mechanism and equations of shear-out failure on bolted connections.
2. Conduct laboratory tests on ultimate shear-out capacities of bolted connections in cold-formed steel sheets to investigate the behaviour of the shear-out failure including the locations of fracture.
3. Create accurate finite element models of bolted connections that can simulate the fracture response.
4. Propose an accurate and consistent design equation based on the laboratory tests and the finite element analysis results.

1.5 Thesis outlines

The outline of each chapter is listed below:

Chapter 1 introduces the importance of an accurate equation for shear-out failure on bolted connections and the controversy around it in the specifications of different countries.

Chapter 2 presents a critical review of existing specifications and literature regarding the mechanism, analysis, and equations of shear-out failure on bolted connections.

Chapter 3 shows the results of 60 tensile tests with varying end distance, edge distance, thickness of steel plate, diameter of bolt hole, steel materials and steel plate cutting method. The load-displacement graphs were drawn and the deformation and fracture location of the steel plate were recorded. The results supported the formulation of the ultimate shear-out capacity.

Chapter 4 presents the FEA model of the bolted connections to show the locations of fracture and “catenary action”. This provides theoretical support for

the equation.

Chapter 5 proposes the equation based on the result of Chapter 3 and uses test data to compare it with different specifications.

Chapter 6 presents the conclusions and proposes works that should be done in the future.

Chapter 2 Literature Review

This chapter provides a critical review of current specifications and literature relating to shear-out failure in the design of bolted connections. The first matter of this part is an overview of current design standards. Following this, the research and proposed equations provided by scholars are discussed. The purpose of the review is to show the variables in these equations and how they influence the capabilities of bolted connections.

2.1 Bolted connection design specifications

2.1.1 Specifications in America

Section J6.1 of the North American Specification for the Design of Cold-Formed Steel Structural Members (AISI 2016) provides the shear-out capacity per bolt of a bolted connection to be:

$$R_n = 1.2 F_u t e_n \quad (1)$$

Equation 1 is based on a shear coefficient of 0.6 which is currently used in AISI specifications. Here e_n is considered to be the net shear length (Teh and Clements 2012); it is the distance between the edge of the bolt hole and the plate end as shown in Figure 2.1.

The earlier North American cold-formed steel specification (AISI 2007) made use of the gross shear length, e_1 defined in Figure 2.1, and gives the shear-out capacity per bolt of a bolted connection to be:

$$R_n = F_u t e_1 \quad (2)$$

which implicitly assumes the shear coefficient to be equal to 0.5. The use of a reduced shear coefficient is required to compensate for the optimistic use of the gross shear plane.

Figure 2.1. Definition of end distance and edge distance

2.1.2 Specifications in Europe

Table 7.4 of the European code EN-1993-1-3:2006 (ECS 2006) specifies the shear-out capacity per bolt to be

$$R_n = 2.5 \alpha_b k_t F_u d_b t \quad (3a)$$

where α_b is the smallest of 1.0 or $e_1/(3d_h)$ and

$$k_t = \begin{cases} \frac{(0.8t+1.5)}{2.5}, & 0.75\text{mm} < t < 1.25\text{mm} \\ 1.0, & t > 1.25\text{mm} \end{cases} \quad (3b)$$

In which d_b is the diameter of the bolt, d_h is the diameter of the bolt hole.

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

2.1.3 Specifications in Australia/New Zealand

Clause 5.2.4.3 of the Australian/New Zealand standard AS/NZS 4600:2018 (SA/SNZ 2018) provides the shear-out capacity per bolt of a bolted connection to be Equation 2.

2.2 Relevant research in the literature

It may be noted that the shear-out failure mode was mentioned by Carril et al. (1994), who tested a large number of bolted connections in cold-reduced steel sheets. However, due to the end distances being more than three times the respective bolt diameters, none of their test specimens failed in shear-out.

Seleim and LaBoube (1996) did an experimental study of single lap bolted connections. The parameters of their study were edge distance, sheet width, sheet thickness, bolt pattern, and percentage elongation. Test results showed that failure modes in low ductility steels are inconsistent with observed failure modes in adequate ductility steels.

Kim and Yura (1999) carried out experimental studies to investigate the effect of ultimate-to-yield ratio on the bearing strength of bolted connections. Two types of steel, one with a high F_u/F_y ratio and the other with a low F_u/F_y ratio were used in the tests. The variables in their experiments were end distance and pitch. They found that the ultimate strength of tear-out failure is more affected by ultimate stress than yield stress and the deformation capacity is more affected by end distance rather than the ultimate-to-yield ratio. The tests showed the Eurocode 3 (1992), or Equation 3 is conservative. The proposed equation is same as Equation 1.

Aalberg and Larsen (2001, 2002) conducted experiments on the shear-out strength of bolted connections in normal strength to high strength steels. Three grades of steel with minimum yield strengths of 355 MPa, 700 MPa and 1100 MPa were used. Both single-bolt and two-bolt connections were included. They stated that the equation specified in ECS (2006) (shown as Equation 3) gives accurate predictions of ultimate load of bearing (shear-out) failure for both normal steels and high strength steels, but the deformation capacity of high strength steel specimens shows a noticeable reduction. They also argued that the 6.35mm

deformation limit is theoretically unclear.

Rogers and Hancock (2000) found that the load-capacity formulation (Equation 2) presented in the American Iron and Steel Institute Specification (AISI 1997) cannot be used to accurately predict the failure modes of thin cold-formed sheet bolted connections loaded in shear. They proposed a modification to the bearing-coefficient provisions to account for the reduced bearing resistance of the connected materials. Hancock (2007) pointed out that the nominal tensile strength and yield stress of G450 sheet steel may be fully utilized in structural design calculations.

Puthli and Fleischer (2001) carried out experimental studies on specimens with varying edge distance, gauge (the distance between the center of two bolts) and plate width. High-strength steel, grade S460, was used in their tests. They found that the reduction of bearing resistance when the edge distance or gauges is less than the requirements in Eurocode 3 (1992), is not necessary. They recommended reduced minimum edge distances and gauge requirements to $e_2=0.9d_0$ and $p_2=1.8d_0$ or at least $e_2=1.0d_0$ and $p_2=2.0d_0$. A reduction factor was proposed to make the equation specified in Eurocode 3 (1992) (shown as Equation 3) accurate.

Chung and Ip (2001) established a finite element model with three-dimensional solid elements to investigate the bearing failure of cold-formed steel bolted connections. The predicted load–extension curves of bolted connections from their model closely followed the measured load–extension curves. The stress–strain curve, contact stiffness and frictional coefficient between element interfaces, and clamping force were shown to be important parameters for prediction of the deformation characteristics of bolted connections. Three distinctive failure modes as observed in lap shear tests were successfully modelled: shear-out failure; bearing failure; and net-section failure. A parametric study on bolted connections with different configurations was performed to provide bearing and shear-out resistances for practical use in design and the results of the finite element modelling are also compared with codified design rules. They found that the design rules of AS/NZS (1996), Equation 2; and Eurocode 3 (1996), Equation 3; are not applicable for bolted connections with high strength steels due to their reduced ductility.

Rex and Easterling (2003) also conducted single bolt bearing tests to examine bolt connection behaviour. Variables in the experiments were end distance, plate width, plate thickness, bolt diameter, edge conditions and material properties. The yield stress F_y of the material they used varied from 299 MPa to 414 MPa. They found that even though most of their specimens failed by bearing, shearing plates rather than sawing them may have a negative effect on the strength.

Xiao and Ishikawa (2005) developed an analytical model for simulating the bearing failure and response characteristics of bolted connections. The analytical model has taken into account the contact conditions at the hole boundary, progressive damage, finite deformation and nonlinear material behaviour. The numerical simulation results for a bolted connection's progressive damage and strength response agreed well with existing experimental data.

Kim and Kuwamura (2007) established finite element (FE) model with three-dimensional solid elements to investigate the structural behaviour of bolted shear connections with thin-walled stainless steel plate. They carried out non-linear material and non-geometric analysis to predict the load–displacement curves of bolted connections. They compared the results of the FE analysis with previous experimental results. It was shown that the numerical approach provided estimates with reasonable accuracy.

Bouchaïr et al. (2008) investigated bolted connection in both stainless steel and carbon steel since these are common in steel structures. A finite element model was developed and validated for the two types of connections. They found that the conventional elastic limit (the greatest stress that can be applied to steel without causing permanent deformation) of austenitic stainless steel is relatively low compared to the ultimate strength.

A total of 50 full-scale connection tests were conducted by Cai and Driver (2010) to investigate the behaviour and strength of bolted connections failed by shear-out and block shear. They stated that the average stress is neither yield stress nor ultimate stress, but a value between these two values. They argued the Europe specification ECS (2001), or Equation 3 gives very conservative results and proposed a better equation for shear failure as follows:

$$R_n = 1.155 e_1 t \left(\frac{F_u + F_y}{2} \right) \quad (4)$$

Moze and Beg (2010; 2014; 2018) conducted single-bolt and multiple-bolt bearing tests to investigate the behaviour and ultimate load of hot-rolled structural steel bolted connections. Finite element analysis also helped to check the equation (Equation 3) specified in Eurocode 3 (1993). A modified bearing resistance coefficient and curve-fitting method were used to get a better equation than that of Eurocode 3 (1993). The equation they proposed for a single-bolted connection failed by shear-out is:

$$R_n = K_1 \alpha_d t d_h F_u \quad (5a)$$

where the coefficients K_1 and α_d consider material grade, bolt position and geometrical parameters:

$$\alpha_d = \frac{e_1}{d_h} \quad (5b)$$

$$K_1 = \begin{cases} \min \left(2.8 \frac{e_2}{d_h} - 1.7; 2.5 \right) & \text{for edge bolts} \\ \min \left(1.4 \frac{e_2}{d_h} - 1.7; 2.5 \right) & \text{for inner bolts} \end{cases} \quad (5c)$$

Salih et al. (2011) investigated the bearing behaviour of stainless steel connections for both thick and thin plates. They developed a numerical model for previously tested specimens in austenitic and ferritic stainless steel. The model was used to perform parametric studies to investigate the variables, including edge distance, end distance, and plate thickness, affecting the bearing failure of bolted connections.

He and Wang (2011) presented an experimental, numerical and analytical study of thin-walled steel bolted connections failed by shear-out. They found that the stiffness and ultimate load could be predicted accurately by using EN 1993-1-3 (ECS 1993), Equation 3. They presented the test results of specimens with four configurations in cold-formed steel sheets. The ratios of end distance to bolt hole diameter tested by them only ranged from 1.0 to 1.5. Based on the results, they proposed a method to calculate the ultimate deformation (hole elongation).

Yu and Panyanouvong (2013) did experimental research to study the bearing

strength of bolted connections with a gap. They found that AISI (2007), or Equation 2 works well for relatively thick connections but provides an unsafe prediction for connections that are relatively thin.

Yang et al. (2013) investigated the load bearing (shear-out) performance of stainless steel bolted connections. They conducted experimental tests on five specimens and adopted FEA models to analyse the effect of end distance, edge distance, thickness of the plate, and diameter of the bolt. Based on the results of the experiments and the FEA model, they proposed an equation for the bearing (shear-out) strength of stainless steel bolted connections. The equation they proposed is:

$$R_n = \left(\frac{0.5e_1}{d_h} + 0.7 \right) F_u d_b t \quad (6)$$

Yan and Young (2013) tested a total of 153 double-shear bolted connection specimens of cold-formed steel. They found that that the strength predicted by the specifications in America (AISI 2007), Equation 2 and Europe (ECS 2006), Equation 3 are generally conservative at elevated temperatures.

Konkong (2017) investigated the ultimate capacities of cold-formed steel bolted connections under shear loading by an experimental program and finite element analysis. The ultimate shear-out capacities were compared with the nominal strengths calculated using the AISI (2012), Equation 1. The results showed that the proposed model, as determined from the experimental investigation, accurately represents the failure mode and ultimate strength of the bolted connection. A modification factor is proposed to make Equation 1 accurate.

Wang et al. (2017) investigated the behaviour of bolted connections in high strength steel and evaluated the effects of end distance, edge distance and steel grade. They found that Eurocode 3 (2005), Equation 3 conservatively predicts the bolt bearing resistance on high strength steel. They also found that splitting failure had a lower resistance value than shear-out failure. The boundary of end distance to edge distance ratio for end splitting failure was verified theoretically and experimentally. Lyu et al. (2019) did a numerical investigation on the bearing (shear-out) behaviour of bolted connections with high strength steels. They proposed an equation for predicting the ultimate bearing (shear-out) load with

consideration of splitting failure. The equation they proposed is:

$$R_n = \left(1.039 \frac{e_1}{d_h} + 0.183\right) F_u d_b t \quad (7)$$

Clements and Teh (2013) used finite element analysis to confirm the active shear plane, shown in Figure 2.2. Nonlinear contact finite brick element analysis results were presented to verify the location of the active shear plane, which was indicated by regions of maximum shear stress. The veracity of the effective shear area was demonstrated through the ability of the resulting block shear equation to predict the failure modes of test specimens, compared with the specifications that assume the gross and the net shear areas.

Figure 2.2. Definition of active shear plane

Teh and Deierlein (2017) pointed out that the inconsistencies of many revisions to the shear-out and block shear requirements of the specifications are primarily related to the assumed unrealistic definition of the net and gross shear planes. A simpler and more accurate block shear design equation based on the active shear plane (same as active shear planes) was proposed. They assumed that the steel on the effective shear planes is fully strain hardened. The single-bolt connection failed by shear-out test results obtained by Moze and Beg (2010, 2014) strongly support the assumption of active shear plane.

Teh and Uz (2015) presented an equation for determining the ultimate shear-out capacity of a structural steel bolted connection and verified it against independent

laboratory test results obtained by other researchers around the world. They also explained the various assumptions embedded in the existing equations, some optimistic and others pessimistic and why certain equations are inaccurate for some particular configurations.

The equation they proposed for determining the ultimate shear-out capacity of a bolted connection is shown below, even though this equation is not suitable for cold-formed steel sheets:

$$R_n = 1.2 t F_u e \quad (8)$$

Elliott et al. (2019) investigated the behaviour and strength of structural steel bolted connections failed by shear yielding and fracture which include shear-out or block shear failure. They verified that using shear failure planes corresponding to the bolt diameter results in significant overestimation of the ultimate load for bolted connections with reduced hole clearance. In contrast, using the effective shear planes is consistently accurate for the specimens with standard, oversize or reduced clearance bolt holes. It is pointed out that the location of fracture initiation can be misidentified by a superficial inspection of the fractured and deformed state of the bolt hole. They also explained that the ultimate shear-out load of a steel bolted connection can be reached without fracture, due to geometric changes downstream of the bolt, provided the reduction in resistance is not offset by strain hardening. This was explained through a finite element analysis that does not simulate fracture, but it is able to determine the ultimate shear-out load of high-strength steel specimens as tested by independent researchers.

2.3 Summary

This chapter discusses the differences between the equations given in Specifications in North America, Europe and Australia/New Zealand. All the code equations assume a linear relationship between the shear-out capacity and the end distance. Equations proposed by researchers are also discussed. It is shown that current Specifications cannot accurately predict the shear-out capacity of a bolted connection in cold-reduced steel sheet; even though Teh and Uz (2015) have already figured out the mechanism of shear-out failure and proposed an equation

which is shown to be accurate for structural steel. An equation is still needed to predict the shear-out capacity of bolted connections in cold-reduced steel sheets.

Chapter 3 Experimental Program

3.1 Introduction

In order to investigate the ultimate load and behaviour of shear-out failure in bolted connections, a total of 71 physical tests were carried out, in which 60 of them failed by shear-out. Test specimens were designed to examine the effects of end distance, edge distance, bolt hole diameter, plate thickness, material properties and cutting method. The primary purpose of the experimental program is to assess the accuracy of the current shear-out equations for cold-formed steel sheets in Section J6.1 of the North American Specification for the Design of Cold-formed Steel Structural Members 2016 (AISI 2016); in Table 7.4 of the European code EN-1993-1-3:2006 (ECS 2006); and in Clause 5.2.4.3 of the Australian/New Zealand standard AS/NZS 4600:2018 (SA/SNZ 2018); or Equations 1, 3 and 2 respectively in this thesis.

The literature review showed that the actual shear plane is the subject of argument by researchers and differs in the specifications of different countries. The active shear plane was proposed by Teh and Uz (2015) in their study of bolted connections failed by shear. Test results can also verify the actual location of the shear plane.

The experimental program consisted of 71 tests, 60 of them failed by shear-out. The tests comprised four nominal bolt diameters, three edge distances, five ratios of end distance to bolt diameter, three thicknesses, two end cutting methods and two sheet steel grades of very different ductility.

3.2 Test materials

Two steel grades, G2 and G450 were chosen for the experimental program. G2 sheet steel is classified as a formability grade, while G450 sheet steel is a structural grade (SA 2011). Material properties of G2 and G450 are obtained from Teh and Uz (2014). The yield stress F_y and ultimate stress F_u of specimens are shown in Table 3.1 and 3.2.

The variable t denotes the base metal thickness without the coating. Tensile loadings of the bolted connection specimens are in the rolling direction of the

sheet steel, as required for structural grade sheet steels (SA 2011).

3.3 Test specimens

Four bolt diameters were used: 8 mm, 10 mm, 12 mm and 16 mm. The bolt holes were drilled 1mm larger than the corresponding bolt diameters. Five ratios of nominal end distance to bolt diameter were tested: 1.25, 1.5, 2.0, 2.5 and 3.0.

According to Moze and Beg (2014), edge distance e_2 tends to influence the ultimate shear load, and three sheet widths were used in the present work: 100 mm, 70 mm and 50 mm. As Rex and Easterling (2003) found that shearing the hot-rolled steel plates tended to result in the end splitting failure before the shear-out failure took place, 23 specimens had their end sheared off rather than saw cut in order to assess its effect on the ultimate capacity.

3.4 Test set-up

All specimens were double shear, single-bolt connections, as shown in Figure 3.1. A 500 kN universal testing machine was used for the test, as shown in Figure 3.2. The critical component is the inner sheet, as each of the two outer plates was 9 mm thick with a yield stress of 585 MPa. The bolts were only finger tightened. All specimens were tested at a stroke rate of 2 mm per minute.

Figure 3.1. Test set-up

Figure 3.2. 500kN universal test machine

3.5 Laboratory test results and discussions

Table 3.1 lists the geometric dimensions of most of the 100 mm wide specimens tested in the present work. All of them failed in shear-out, as shown in Appendix A. The load-displacement graphs are given in Appendix B. An empty cell in a table indicates that the data in the above cell applies.

Table 3.1 also shows the professional factors of the cold-formed steel code equations for determining the shear-out capacity of bolted connections in cold-reduced steel sheets. A professional factor PF is the ratio of the ultimate test load P_t to the predicted capacity R_n . The professional factors of Equations 1 to 3 are listed in tables as PF_{AISI} , $PF_{AS/NZS}$ and PF_{ECS} respectively. The professional factors of Equation 8 are listed as PF_{so} . It can be seen that all the code equations are too conservative for the present specimens.

It can be seen in Table 3.1 that the professional factor of Equation 1 generally increases with decreasing end distance (other variables remaining the same). This observation suggests that catenary action contributes to the shear-out capacity of a bolted connection with a short end distance such as that shown in Figure 3.3. As

indicated in Figure 3.4, the shear-out load imposed by the bolt is resisted by the shear stresses and the catenary stresses. The suggestion is corroborated using finite element analysis in Chapter 4.

Figure 3.3. Specimen with a short end distance

Figure 3.4. Catenary action

It should be noted that the trend in the professional factors of Equation 1 is not necessarily consistent with that in the others because the other code equations do not capture the actual values of the bolt diameters, and hence the shear areas actually available.

Table 3.1 Results for bolted connections with different end distance

Specimen	W (mm)	d_h (mm)	e_I (mm)	t (mm)	F_y (MPa)	F_u (MPa)	P_t (KN)	PF_{AISI}	$PF_{AS/NZS}$	PF_{ECS}	PF_{so}
HX005	100	9	24	1.45	320	400	15.9	1.18	1.14	1.77	1.05
HX004	-	-	20	-	-	-	13.7	1.24	1.16	1.78	1.08
HX003	-	-	16	-	-	-	11.4	1.40	1.21	1.87	1.18
HX002	-	-	12	-	-	-	8.4	1.68	1.24	1.91	1.28
HX001	-	-	10	-	-	-	6.8	1.66	1.13	1.74	1.20
HX010	-	13	36	-	-	-	22.8	1.13	1.11	1.57	1.02
HX009	-	-	30	-	-	-	17.7	1.09	1.03	1.46	0.96
HX008	-	-	24	-	-	-	15.6	1.28	1.12	1.58	1.08
HX007	-	-	18	-	-	-	11.9	1.47	1.13	1.60	1.15
HX006	-	-	15	-	-	-	9.7	1.61	1.10	1.57	1.17
HX015	-	11	30	2.35	310	390	30.4	1.14	1.12	1.64	1.02
HX014	-	-	25	-	-	-	25.2	1.19	1.11	1.63	1.04
HX013	-	-	20	-	-	-	20.2	1.27	1.10	1.62	1.06
HX012	-	-	15	-	-	-	15.9	1.51	1.15	1.69	1.18
HX011	-	-	12.5	-	-	-	12.9	1.62	1.10	1.62	1.17
HX020	-	17	48	-	-	-	46.3	1.07	1.06	1.44	0.97
HX019	-	-	40	-	-	-	40.8	1.15	1.09	1.49	1.02
HX018	-	-	32	-	-	-	33.9	1.28	1.13	1.54	1.09
HX017	-	-	24	-	-	-	25.5	1.45	1.13	1.54	1.14
HX016	-	-	20	-	-	-	20.5	1.55	1.09	1.48	1.15
HX025	-	13	36	1.45	550	590	30.8	1.01	0.99	1.41	0.91
HX024	-	-	30	-	-	-	27.1	1.12	1.05	1.49	0.98
HX023	-	-	24	-	-	-	21.9	1.22	1.07	1.51	1.03
HX022	-	-	18	-	-	-	16.3	1.38	1.06	1.50	1.08
HX021	-	-	15	-	-	-	13.5	1.49	1.03	1.46	1.09
HX029	-	11	24	2.95	520	555	43.7	1.16	1.08	1.58	1.01
HX028	-	-	20	-	-	-	34.9	1.27	1.09	1.60	1.06
HX027	-	-	15	-	-	-	26	1.42	1.07	1.57	1.10
HX026	-	-	12.5	-	-	-	21.9	1.59	1.07	1.57	1.14
HX034	-	17	40	-	-	-	71.1	1.15	1.09	1.48	1.02
HX033	-	-	32	-	-	-	58.3	1.28	1.12	1.53	1.08
HX032	-	-	24	-	-	-	43	1.44	1.11	1.51	1.12
HX031	-	-	20	-	-	-	36.7	1.68	1.14	1.55	1.22

Table 3.2 compares the results of shear-out specimens having different widths, and hence different edge distances, with the other variables remaining constant. There is no significant effect of the edge distance on the professional factors, justifying the absence of the variable e_2 in the existing cold-formed steel equations for shear-out. Figure 3.5 compares the load-displacement graphs between specimens having the same material and geometric variables except for their edge distances. It can be seen that the edge distance e_2 had little effect on the responses of the specimens, which had relatively wide edge distances.

Table 3.2 Results for bolted connections with different edge distance

Specimen	W (mm)	d_h (mm)	e_1 (mm)	t (mm)	F_y (MPa)	F_u (MPa)	P_t (KN)	PF_{AISI}	$PF_{AS/NZS}$	PF_{ECS}	PF_{so}
HX011	100	11	12.5	2.35	310	390	12.9	1.62	1.10	1.62	1.17
HX036	70	-	-	-	-	-	12.1	1.62	1.06	1.54	1.14
HX060	50	-	-	-	-	-	13	1.65	1.10	1.60	1.18
HX012	100	-	15	-	-	-	15.9	1.51	1.15	1.69	1.18
HX061	50	-	-	-	-	-	15.1	1.45	1.08	1.58	1.11
HX013	100	-	20	-	-	-	20.2	1.27	1.10	1.62	1.06
HX037	70	-	-	-	-	-	20.6	1.30	1.11	1.62	1.08
HX062	50	-	-	-	-	-	20.4	1.27	1.10	1.61	1.07
HX015	100	-	30	-	-	-	30.4	1.14	1.12	1.64	1.02
HX038	70	-	-	-	-	-	28.7	1.08	1.05	1.52	0.97
HX016	100	17	20	-	-	-	20.5	1.55	1.09	1.48	1.15
HX039	70	-	-	-	-	-	19.5	1.60	1.08	1.46	1.15
HX018	100	-	32	-	-	-	33.9	1.28	1.13	1.54	1.09
HX040	70	-	-	-	-	-	31.3	1.24	1.08	1.47	1.04
HX016	100	-	20	-	-	-	20.5	1.55	1.09	1.48	1.15
HX063	50	-	-	-	-	-	20.7	1.65	1.12	1.51	1.19
HX017	100	-	24	-	-	-	25.5	1.45	1.13	1.54	1.14
HX064	50	-	-	-	-	-	23.8	1.42	1.09	1.47	1.11
HX026	100	11	12.5	2.95	520	555	21.9	1.59	1.07	1.57	1.14
HX042	70	-	-	-	-	-	21.2	1.61	1.05	1.52	1.13
HX066	50	-	-	-	-	-	21.3	1.58	1.03	1.50	1.11
HX027	100	-	15	-	-	-	26	1.42	1.07	1.57	1.10
HX067	50	-	-	-	-	-	26.3	1.37	1.04	1.52	1.06
HX028	100	-	20	-	-	-	34.9	1.27	1.09	1.60	1.06
HX043	70	-	-	-	-	-	36	1.28	1.09	1.59	1.06
HX068	50	-	-	-	-	-	36.3	1.26	1.09	1.59	1.06
HX031	100	17	-	-	-	-	36.7	1.68	1.14	1.55	1.22
HX045	70	-	-	-	-	-	34.3	1.59	1.06	1.44	1.14
HX069	50	-	-	-	-	-	35.7	1.59	1.07	1.45	1.14

(a)

(b)

Figure 3.5. Load-displacement graphs: (a) HX011 vs. HX060 (b) HX013 vs. HX062

The response of Specimen HX064, which had a narrow edge distance equal to about 1.5 times the bolt diameter, was significantly more flexible than Specimen HX017, as shown in Figure 3.6. The two specimens were nominally identical to each other except for their edge distances. However, the professional factor of Equation 1 for Specimen HX064 was only 2% lower than for Specimen HX017, despite the edge distance of the former being half that of the latter.

Figure 3.6. Load-displacement graphs of HX017 and HX064

The specimens in Table 3.3 and all the 70-mm wide specimens in Table 3.2 had been sheared off rather than saw cut, and failed in shear-out like their sawn counterparts. It therefore appears that shearing off does more damage to thicker hot-rolled steel plates (Rex and Easterling 2003), often changing their failure mode from shear-out to end splitting.

Table 3.3 Results for bolted connections with their end sheared-off

Specimen	W (mm)	d_h (mm)	e_l (mm)	t (mm)	F_y (MPa)	F_u (MPa)	P_t (kN)	PF_{AISI}	$PF_{AS/NZS}$	PF_{ECS}	PF_{so}
HX050	100	11	30	2.35	310	390	31.4	1.15	1.12	1.63	1.03
HX049	-	-	20	-	-	-	21.8	1.36	1.17	1.69	1.13
HX048	-	-	12.5	-	-	-	13.5	1.77	1.16	1.68	1.25
HX053	-	17	48	-	-	-	47.6	1.09	1.07	1.45	0.98
HX052	-	-	32	-	-	-	33.8	1.30	1.14	1.54	1.10
HX051	-	-	20	-	-	-	21.2	1.68	1.14	1.54	1.21
HX055	-	11	-	2.95	520	555	37.2	1.28	1.11	1.61	1.07
HX054	-	-	12.5	-	-	-	22.4	1.62	1.07	1.55	1.15
HX059	-	17	48	-	-	-	84	1.08	1.06	1.43	0.97
HX058	-	-	32	-	-	-	59.4	1.28	1.12	1.51	1.08

However, for a given configuration, shearing off the cold-reduced sheet steel specimen did increase the likelihood of the end splitting mode, as shown in Figure 3.7 for Specimen HX057. The professional factor is about 5% lower than that of the sawn cut Specimen HX031, which was otherwise nominally identical.

Figure 3.7 End splitting failure

Chapter 4 Finite Element Analysis

4.1 Introduction

As noted in the preceding section, catenary action appears to contribute to the shear-out capacity of a bolted connection with a short end distance. This effect is not easy to verify by experiment. However, finite element analysis, which can show the stresses of specimens, may help to verify the catenary action. Specimens HX011 and HX015 were selected for investigation owing to their respective end distances, shown in Table 3.1.

As discussed in Chapter 3, the literature review showed that the actual location of the shear plane is the subject of argument by researchers and differs in the specifications of different countries. FEA can be used to verify the actual location of a shear plane since the location of stress can be observed from the FEA model. The finite element analysis software ABAQUS (2014) was used for the model simulation.

4.2 Model of specimens

4.2.1 Assumptions of the model

To simplify the analysis, the following assumptions are applied:

1. The bolt will not fail; the deformation of the bolt can be ignored.
2. The friction between the outer plate and the specimen can be ignored.
3. The material of the plate is isotropic. (It is actually anisotropic; this thesis only investigates the strength of the specimens in the loading direction, so the material is defined as isotropic in the model.)

4.2.2 Assembly of the model

The geometry of the model was defined as two parts, plate and bolt, positioned relative to one another in an assembly. The part plate was modeled as a deformable body with a hole. The part bolt was modeled as a 3D analytical rigid body revolved shell based on the Assumption 1 in Section 4.2.1. Washers were not used in the test, so they were not modeled. The outer plate was not modeled based on the Assumption 2 in Section 4.2.1 even though the deformation of the plate in the outer plane was limited. Advantage was taken of the symmetry by modeling only a quarter of the inner sheet. The mirror images of the symmetric model were later combined to illustrate the stress contours. The assembly of the model is shown in Figure 4.1.

Figure 4.1 Assembly of the model (quarter model)

4.2.3 Material properties of the model

The steel for HX011 and HX015 is G2 with a thickness of 2.4 mm. The elastic modulus was assumed to be 200 GPa, and the Poisson's ratio was 0.3. The plasticity of the steel material was handled through the von Mises yield criterion. Table 4.1 shows the true stress-strain curve used in the simulation, generated using an engineering yield stress of 310 MPa, a tensile strength of 390 MPa, and a strain of 0.4 at the ultimate engineering stress.

Table 4.1 Stress-strain curve of HX011 and HX015

True stress(MPa)	True plastic strain
248	0
328	0.006
338	0.011
349	0.019
362	0.032
378	0.053
400	0.088
431	0.141
477	0.221
546	0.334
707	0.6

Fracture initiation and propagation were simulated in the present analysis in order to trace the post-ultimate path of each connection so that the finite element results could be validated. Fracture initiation was simulated using the shear criterion of progressive damage for ductile metals (ABAQUS 2014). Fracture propagation was simulated using the effective plastic displacement based damage evolution option in ABAQUS (2014), which assumes that damage is characterized by progressive degradation of the material stiffness leading to material failure. The phenomenological model assumes that the equivalent plastic strain at the onset of damage is a function of the shear stress ratio. The stiffness of a damaged element could degrade by up to 99 percent, but the element deletion option was kept inactive to avoid convergence issues. The equivalent plastic displacement at failure was set to be 0.45 and the material parameter k_s in ABAQUS 6.14 Standard (ABAQUS 2014) was -0.2. The methodology developed by Ahmed et al. (2019) was used to obtain the shear damage parameters in Table 4.2. Implicit solver was used.

Table 4.2 Shear damage parameters of HX011 and HX015

Fracture strain	Shear stress ratio
3	1.65
0.39	1.731
0.4	1.732
0.41	1.733
3	1.8

4.2.4 Load and boundary conditions of the model

The top set of the plate was fixed. Loading of the bolted connection was simulated by displacing the bolt, which would be resisted by the surface contact with the bolt hole at the downstream end, in the same manner as conducted by Clements and Teh (2013). In the contact simulation, the tangential behaviour was defined using the penalty formulation, and a friction coefficient of 0.1 was used. It only influenced the resistance and had only a small impact on the angle of the load-displacement curve. As only a quarter of the model was built, the deformation in the planes of symmetry was limited.

4.2.5 Mesh of the model

The inner sheet of each double-shear connection was modeled using hexahedral reduced integration brick element C3D8R available in ABAQUS Standard (ABAQUS 2014). The thickness of the specimen was in general meshed with four finite elements as shown in Figure 4.2. In the zone of large stress gradients, the size of the mesh was defined as 0.5 mm, while elsewhere it was defined as 1 mm.

Figure 4.2 Mesh of the model

4.2.6 Validity of the model

It can be seen from Figure 4.3 that the present models are able to closely simulate the responses of Specimens HX011 and HX015. There is very good agreement between the ultimate test loads and the ultimate limit loads, and their corresponding displacement. Figure 4.4 and 4.5 also showed that the location of the fracture and the deformation of the FEA model gave good agreement with the test result.

(a)

(b)

Figure 4.3. Load-displacement graphs of FEA and test result: (a) HX011. (b) HX015

(a)

(b)

Figure 4.4 Location of fracture and deformation of HX011: (a) Test result, (b) FEA model

(a)

(b)

Figure 4.5 Location of fracture and deformation of HX015: (a) Test result, (b) FEA model

4.3 FEA results and discussions

Figure 4.6 shows the shear stress contour of HX011. It can be seen the actual shear plane is neither net shear plane nor gross shear plane but matches the “active shear plane,” which was proposed by Clements and Teh (2013).

Figure 4.6 Shear stress contour of HX011

Figure 4.7(a) and 4.7(b) show that the transverse normal stresses associated with the catenary action, plotted at the displacements corresponding to the respective ultimate test loads, were significantly higher in Specimen HX011 than in Specimen HX015. The maximum transverse normal stress in the narrower strip was 566 MPa at the ultimate load of 12.9 kN, while that in the wider strip was 487 MPa at the ultimate load of 30.4 kN. The maximum catenary force downstream of the bolt in the narrower strip was 4.28 kN at the load of 12.9 kN, while the other was only 3.37 kN at the same load. Importantly, the angle θ at the ultimate limit state, defined in Figure 3.4, of Specimen HX011 was 33.9 degrees, while that of Specimen HX015 was only 19.9 degrees. The present finite element results therefore corroborate the inference made in the preceding section that the catenary effect contributes significantly to the shear-out capacity of a bolted connection with a short end distance, and dissipates with increasing end distances.

(a)

(b)

Figure 4.7 Transverse stress contours: (a) HX011, (b) HX015

4.4 Summary

In this chapter, the finite analysis model is built. The validity of the model is verified by matching the load-displacement graphs and comparing the deformation and location of fracture in the FEA models and test specimens. The numerical simulation has shown that catenary action in a narrow strip downstream of the bolt has a significant effect on the shear-out capacity. The effective shear plane is also verified by the FEA model.

Chapter 5 Proposed Shear-out Equation

Equations for prediction of the ultimate load of bolted connections in cold-reduced steel sheets specified in different standards are different from each other not only in coefficient but also in variables. An accurate and consistent equation is needed. This chapter will discuss the variables embedded in the equations and propose an equation which can be used in the industry.

5.1 Discussion of current equations

The equation specified in AISI (2016) is shown as Equation 1 in Chapter 2. It can be seen in Tables 3.1 to 3.3 that the equation is conservative as the mean value of the professional factor is 1.37, that is, it underestimates the capacity of specimens by 37%. The use of the clear distance e_n , as shown in Figure 2.1, is an approximation since the two shear failure planes cannot coincide with the centreline of the bolt hole, or with each other (Teh and Uz 2015).

The earlier North American cold-formed steel specification (AISI 2007) made use of the nominal end distance e_1 , Equation 2. The use of a reduced shear coefficient is required to compensate for the optimistic use of the gross shear plane indicated by the nominal end distance e_1 . Tables 3.1 to 3.3 show it underestimates the capacity of specimens by 10%. Equation 2 is still current in the Australian/New Zealand standard (AS/NZS 2018), and will henceforth be referred to as the Australian/New Zealand equation.

The equation specified in ECS (2006) is given as Equation 3 in Chapter 2. Tables 3.1 to 3.3 show that the ECS equation underestimates the capacity of specimens by 57%.

5.2 Discussion of variables

5.2.1 Edge distance

The test results of HX011 vs. HX060, HX013 vs. HX062 and HX017 vs. HX064 discussed in Chapter 3 show that edge distance has insignificant influence (no more than 3% difference) on the ultimate load of shear-out failure. So the influence of the edge distance is not considered in the equation proposed by this thesis.

5.2.2 End distance

Discussions in Chapter 2 show that all the specifications and equations proposed by researchers recognized the end distance as an important influence factor. Net shear plane and gross shear plane are embedded in the current specifications. The test results and FEA show that the actual shear plane is the effective shear plane, as a result the e shown in Figure 2.1 is used.

The test results also show the professional factors of Equation 1 generally increase with decreasing clear end distance as a result of catenary action. The finite element analysis can also verify this action. A coefficient to show the catenary action will be proposed in the equation.

5.2.3 Cutting method of the plate

Rex and Easterling (2003) found that shearing plates rather than sawing may have a negative effect on the strength. As discussed in Chapter 3, test results show that shearing off increases the likelihood of the end splitting mode, which may decrease the capacity by 5%. It will not be considered in the equation.

5.3 Proposed shear-out equation

Equation (8) has been demonstrated by Teh and Uz (2015) to be accurate for hot-rolled steel specimens tested by independent researchers. The use of the effective shear planes depicted in Figure 2.2 has also been comprehensively validated by Elliott et al. (2019) through finite element analysis incorporating shear fracture simulation. It can be seen from the Table 3.1 to 3.3 that as with Equation (1), the professional factors of Equation (8) generally increase with decreasing end distances. It is nevertheless accurate for specimens with relatively long end distances.

Equation (9a) therefore forms the basis of the equation proposed in the present work for cold-reduced sheet steel bolted connections

$$R_n = 1.2 c F_u t e \quad (9a)$$

in which the parameter c is the coefficient accounting for the contribution of the catenary action depicted in Figure 3.4.

As the end distance increases to a threshold value, the shear-out failure mode changes to the bearing failure mode. At this point, the catenary effect vanishes completely. The threshold value of the nominal end distance e_1 was found by Teh and Uz (2016) to be 3.14 times the bolt diameter for a structural steel connection. For the present work, it is assumed to be three times the bolt diameter Equation (9a) becomes

$$R_n = 1.2 \left(\frac{3d_b}{e_1} \right)^p F_u t e \quad (9b)$$

in which the power p is determined empirically from the laboratory test results. It was found that the use of a power equal to one fifth led to reasonable results

$$R_n = 1.2 \left(\frac{3d_b}{e_1} \right)^{1/5} F_u t e \quad (9c)$$

The professional factors of Equation 5, Equation 7, Equation 8 and Equation 9 are listed as PF_{Moze} , PF_{Tongji} , PF_{so} and PF_P respectively in Table 5.1, while the professional factors of code equations (Equation 1, Equation 2 and Equation 3) are listed as PF_{AISI} , $PF_{AS/NZS}$ and PF_{ECS} . It can be seen that the proposed equation is significantly more accurate than most of the cold-formed steel code equations. It should be noted that Equation 5 and Equation 7 also gave a good result not only in mean value but in COV. However, the max value of the professional factor of Equation 5 (1.19) is too large which shows the equation is over-conservative and the minimum value of the professional factor of Equation 7 (0.86) is too small which shows the equation is unsafe. Both of them used the curve-fitting method to get the equation, which is theoretically unclear.

Table 5.1 Professional factor of proposed equation vs. other equations

Specimen	PF_{so}	PF_{Moze}	PF_{Tongji}	PF_{AISI}	PF_{ECS}	$PF_{AS/NZS}$	PF_p
HX001	1.20	1.01	0.94	1.66	1.74	1.13	1.01
HX002	1.28	1.12	1.05	1.68	1.91	1.24	1.11
HX003	1.18	1.09	1.06	1.40	1.87	1.21	1.09
HX004	1.08	1.04	1.03	1.24	1.78	1.16	1.05
HX005	1.05	1.03	1.03	1.18	1.77	1.14	1.05
HX006	1.17	1.02	0.92	1.61	1.57	1.10	0.99
HX007	1.15	1.04	0.97	1.47	1.60	1.13	1.00
HX008	1.08	1.03	0.98	1.28	1.58	1.12	0.99
HX009	0.96	0.95	0.92	1.09	1.46	1.03	0.92
HX010	1.02	1.02	1.00	1.13	1.57	1.11	1.01
HX011	1.17	1.01	0.92	1.62	1.62	1.10	0.99
HX012	1.18	1.05	0.98	1.51	1.69	1.15	1.03
HX013	1.06	1.00	0.97	1.27	1.62	1.10	0.98
HX014	1.04	1.01	0.99	1.19	1.63	1.11	1.00
HX015	1.02	1.02	1.01	1.14	1.64	1.12	1.02
HX016	1.15	1.03	0.92	1.55	1.48	1.09	0.97
HX017	1.14	1.07	0.97	1.45	1.54	1.13	1.00
HX018	1.09	1.07	1.00	1.28	1.54	1.13	1.01
HX019	1.02	1.03	0.98	1.15	1.49	1.09	0.98
HX020	0.97	1.00	0.96	1.07	1.44	1.06	0.97
HX021	1.09	0.95	0.86	1.49	1.46	1.03	0.92
HX022	1.08	0.98	0.90	1.38	1.50	1.06	0.94
HX023	1.03	0.98	0.94	1.22	1.51	1.07	0.95
HX024	0.98	0.97	0.94	1.12	1.49	1.05	0.95
HX025	0.91	0.92	0.90	1.01	1.41	0.99	0.91
HX026	1.14	0.98	0.89	1.59	1.57	1.07	0.96
HX027	1.10	0.98	0.91	1.42	1.57	1.07	0.95
HX028	1.06	0.99	0.96	1.27	1.60	1.09	0.97
HX029	1.01	0.98	0.96	1.16	1.58	1.08	0.97
HX031	1.22	1.08	0.95	1.68	1.55	1.14	1.02
HX032	1.12	1.05	0.95	1.44	1.51	1.11	0.98
HX033	1.08	1.06	0.99	1.28	1.53	1.12	0.99

Table 5.1 (continued)

Specimen	PF_{so}	PF_{Moze}	PF_{Tongji}	PF_{AISI}	PF_{ECS}	$PF_{AS/NZS}$	PF_P
HX034	1.02	1.03	0.97	1.15	1.48	1.09	0.98
HX036	1.14	0.99	0.88	1.62	1.54	1.06	0.95
HX037	1.08	1.05	0.97	1.30	1.62	1.11	0.99
HX038	0.97	0.99	0.95	1.08	1.52	1.05	0.96
HX039	1.15	1.07	0.90	1.60	1.46	1.08	0.96
HX040	1.04	1.07	0.95	1.24	1.47	1.08	0.96
HX042	1.13	0.98	0.87	1.61	1.52	1.05	0.94
HX043	1.06	1.03	0.96	1.28	1.59	1.09	0.97
HX045	1.14	1.05	0.88	1.59	1.44	1.06	0.95
HX048	1.25	1.06	0.96	1.77	1.68	1.16	1.04
HX049	1.13	1.07	1.02	1.36	1.69	1.17	1.04
HX050	1.03	1.03	1.01	1.15	1.63	1.12	1.03
HX051	1.21	1.08	0.95	1.68	1.54	1.14	1.01
HX052	1.10	1.08	1.00	1.30	1.54	1.14	1.01
HX053	0.98	1.02	0.97	1.09	1.45	1.07	0.98
HX054	1.15	0.98	0.89	1.62	1.55	1.07	0.96
HX055	1.07	1.01	0.97	1.28	1.61	1.11	0.99
HX058	1.08	1.06	0.98	1.28	1.51	1.12	0.99
HX059	0.97	1.00	0.96	1.08	1.43	1.06	0.96
HX060	1.18	1.08	0.91	1.65	1.60	1.10	0.97
HX061	1.11	1.06	0.92	1.45	1.58	1.08	0.95
HX062	1.07	1.08	0.97	1.27	1.61	1.10	0.96
HX063	1.19	1.19	0.93	1.65	1.51	1.12	0.98
HX064	1.11	1.15	0.93	1.42	1.47	1.09	0.95
HX066	1.11	1.01	0.86	1.58	1.50	1.03	0.93
HX067	1.06	1.02	0.89	1.37	1.52	1.04	0.93
HX068	1.06	1.07	0.96	1.26	1.59	1.09	0.97
HX069	1.14	1.14	0.90	1.59	1.45	1.07	0.96
P_m	1.09	1.03	0.95	1.37	1.57	1.10	0.98
Cov	0.071	0.049	0.050	0.150	0.066	0.040	0.039
Max	1.28	1.19	1.06	1.77	1.91	1.24	1.11
Min	0.91	0.92	0.86	1.01	1.41	0.99	0.91

Equation 9 was verified against the laboratory test results of He and Wang (2011), which to the authors' knowledge are the only independent shear-out test results available in the literature for cold-reduced steel sheets. Their specimens had 2 mm hole clearance.

Table 5.2 shows that the proposed equation is quite accurate for the four configurations tested by He and Wang (2011) including Specimen D14-14-28, which had an end distance equal to the bolt hole diameter, and therefore had the greatest catenary effect of all specimens studied in the present work. All the code equations, on the other hand, are quite conservative for the specimens tested by He and Wang (2011), consistent with their results for the specimens tested by the author.

Table 5.2 Test results of He and Wang (2011)

Specimen	W (mm)	d_h (mm)	e_1 (mm)	t (mm)	F_u (MPa)	PF_{so}	PF_{Moze}	PF_{Tongji}	PF_{AISI}	PF_{ECS}	$PF_{AS/NZS}$	PF_P
D14-14-28	56	14	14	1.5	320	1.23	1.1	0.9	1.84	1.54	1.1	1.01
D14-20-80	108	-	20	-	-	1.26	1.2	1.1	1.6	1.75	1.25	1.12
D14-28-80	-	-	28	-	-	1.13	1.1	1.1	1.32	1.66	1.19	1.08
D14-21-42	60	-	21	-	-	1.15	1.1	1	1.44	1.61	1.15	1.03

5.4 Resistance factor

The mean professional factors P_m given by the code and proposed equations for the 64 specimens listed are given in Table 5.3, along with their coefficients of variation.

Table 5.3 Performance of alternative equations for 64 specimens

Specimen	PF_{so}	PF_{Moze}	PF_{Tongji}	PF_{AISI}	PF_{ECS}	$PF_{AS/NZS}$	PF_P
P_m	1.10	1.04	0.95	1.38	1.57	1.10	0.99
Cov	0.072	0.051	0.053	0.15	0.065	0.044	0.043

Section K2.1.1 of the North American specification (AISI 2016) specifies that the resistance factor ϕ of a design equation is determined as follows

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

in which C_ϕ is the calibration coefficient equal to 1.52 in the case of the Load and Resistance Factor Design (LRFD), M_m is the mean value of the material factor equal to 1.10 according to Table K2.1.1-1 of the North American specification, F_m is the mean value of the fabrication factor equal to 1.00, and P_m is the mean value of the professional factor, which is equal to 0.99 for the proposed equation

as shown in Table 5.3.

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

$$P = -\beta_0 \sqrt{V_M^2 + V_F^2 + C_P V_P^2 + V_Q^2} \quad (11)$$

in which V_M is the coefficient of variation of the material factor equal to 0.08 in the present case, V_F is the coefficient of variation of the fabrication factor equal to 0.05, V_P is the coefficient of variation of the professional factor (equal to 0.043 for the proposed equation), C_P is the correction factor equal to 1.01 as computed from the relevant equation given in Section K2.1.1, and V_Q is the coefficient of variation of load effects equal to 0.21 as specified in Section K2.1.1.

It was found that in order to achieve the target reliability index β_0 of 3.5 in the LRFD, Equation 10 yields a resistance factor of 0.70 for the proposed Equation 9.

Chapter 6 Conclusions

6.1 Conclusions

This thesis has pointed out that catenary action contributes significantly to the shear-out capacity of a bolted connection having a short end distance. The catenary effect dissipates with increasing end distances until it vanishes at the threshold end distance where the shear-out failure mode changes to the bearing failure mode. Therefore, the shear-out capacity does not vary linearly with the end distance. The North American, European and Australian/New Zealand cold-formed steel code equations do not account for the varying catenary effect over end distance and significantly underestimate the ultimate shear-out capacity.

The edge distance does not have a significant effect on the shear-out capacity and is therefore not a parameter in the proposed shear-out equation. Material ductility was not found to have a noticeable impact on the shear-out capacities of the test specimens, which were loaded in the rolling direction of the sheet steel in accordance with the standard. While shearing off rather than saw cutting a cold-reduced steel sheet may change the failure mode from shear-out to end splitting, the effect is not so pronounced as for hot-rolled steel plates.

A simple refinement to an existing equation is able to accurately capture the catenary effect. The proposed design equation is dimensionally consistent and transparent. The equation is reasonably accurate for the cold-reduced sheet steel specimens tested by the authors and those reported in the literature, which had bolt diameters ranging from 8 mm to 16 mm, and end distances ranging from 1.0 to 3.0 times the bolt diameter.

It is recommended that a resistance factor of 0.70 be applied to the proposed design equation in the LRFD approach of the North American specification for the design of cold-formed steel structures.

6.2 Future work

The present work focuses on the one-bolt connection failed by shear-out. It can be seen from the conclusion that catenary action has a significant impact on the ultimate shear load of one-bolt connections; its effects on two-bolt and multiple-

bolt connections have not been discussed in this thesis.

As mentioned in Chapter 3, shearing rather than sawing cutting may change the failure mode from shear-out to end splitting. The mechanism of the mode change should also be investigated. The test results also showed that the proposed equation can coincidentally predict the ultimate load of specimens failed by splitting even though it is a different failure mode. The mechanism of end splitting failure should be investigated in future research.

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Appendices

Appendix A Photographs of tested specimens

A.1 Specimens failing in shear-out:

HX001

HX002

HX003

HX004

HX005

HX006

HX007

HX008

HX009

HX010

HX011

HX012

HX013

HX014

HX015

HX016

HX017

HX018

HX019

HX020

HX021

HX022

HX023

HX024

HX025

HX026

HX027

HX028

HX029

HX031

HX032

HX033

HX034

HX036

HX037

HX038

HX039

HX040

HX042

HX043

HX045

HX048

HX049

HX050

HX051

HX052

HX053

HX054

HX055

HX058

HX059

HX060

HX061

HX062

HX063

HX064

HX066

HX067

HX068

HX069

A.2 Specimens failing in end split:

HX035

HX046

HX047

HX057

HX070

A.3 Specimens failing in other modes (net section fracture or bolt shear)

HX041

HX065

HX071

HX030, HX044 and HX056

Appendix B Load-deflection curves of specimens failing in shear-out

Figure B.1 Load-deflection curves of 1.5 mm thick 100 mm wide G2 sheet specimens with 8 mm bolt

Figure B.2 Load-deflection curves of 1.5 mm thick 100 mm wide G2 sheet specimens with 12 mm bolt

Figure B.3 Load-deflection curves of 2.4 mm thick 100 mm wide G2 sheet specimens with 10 mm bolt

Figure B.4 Load-deflection curves of 2.4 mm thick 100 mm wide G2 sheet specimens with 16 mm bolt

Figure B.5 Load-deflection curves of 1.5 mm thick 100 mm wide G450 sheet specimens with 12 mm bolt

Figure B.6 Load-deflection curves of 3 mm thick 100 mm wide G450 sheet specimens with 10 mm bolt

Figure B.7 Load-deflection curves of 3 mm thick 100 mm wide G450 sheet specimens with 16 mm bolt

Figure B.8 Load-deflection curves of 2.4 mm thick 70 mm wide G2 sheet specimens with 10 mm bolt

Figure B.9 Load-deflection curves of 2.4 mm thick 70 mm wide G2 sheet specimens with 16 mm bolt

Figure B.10 Load-deflection curves of 3 mm thick 70 mm wide G450 sheet specimens with 10 mm bolt

Figure B.11 Load-deflection curve of 3 mm thick 70 mm wide G450 sheet specimens with 16 mm bolt

Figure B.12 Load-deflection curves of 2.4 mm thick 100 mm wide G2 sheet specimens with 10 mm bolt

Figure B.13 Load-deflection curves of 2.4 mm thick 100 mm wide G2 sheet specimens with 16 mm bolt

Figure B.14 Load-deflection curves of 3 mm thick 100 mm wide G450 sheet specimens with 10 mm bolt

Figure B.15 Load-deflection curves of 3 mm thick 100 mm wide G450 sheet specimens with 16 mm bolt

Figure B.16 Load-deflection curves of 2.4 mm thick 50 mm wide G2 sheet specimens with 10 mm bolt

Figure B.17 Load-deflection curves of 2.4 mm thick 50 mm wide G2 sheet specimens with 16 mm bolt

Figure B.18 Load-deflection curves of 3 mm thick 50 mm wide G450 sheet specimens with 10 mm bolt

Figure B.19 Load-deflection curve of 3 mm thick 50 mm wide G450 sheet specimens with 16 mm bolt