

Fracture Behaviour of Cold-Formed Steel Bolted Connections

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ABSTRACT

In this thesis, research into fracture behaviour and capacity of cold-formed steel (CFS) bolted connections is presented based on three independent experimental programs in conjunction with a numerical study using Finite Element (FE) simulation. Current design rules are also discussed and new design models are proposed.

In terms of the first experimental program, testing bolted connections with staggered holes under monotonic shear loading was conducted to provide insights into the fracture behaviour and capacity of the connections failing in block shear. Test results are used to propose a new design formula for block shear strength accounting for the combined actions of shear and tension ruptures in the staggered zones which is not available in the current Australian and New Zealand Standard (AS/NZS 4600:2018) and North American Specification (AISI S100:16 (R2020) w/S3-22). A calibrated finite element model (FEM) was then validated against the test results to achieve reliable data using a parametric procedure. The large data set is used for the verification procedure of the proposed formula. In the finite-element (FE) simulation, to study the whole range of deformation of the staggered hole bolted connections (i.e., elastic, plastic and fracture stages), a plasticity model and a ductile damage model are applied to the CFS sheets to represent various stress states in the steel during failure. Reliability analyses are employed to evaluate the capacity reduction/resistance factors of the proposed model for Load and Resistance Factor Design (LRFD) and Limit State Design (LSD).

Further, in the second experimental program, testing shear beam-end bolted connections joining CFS channels to a tubular column was performed. A dual actuator test apparatus was used to create realistic loading combinations of shear and rotation to investigate their effects on the fracture behaviour and block shear capacity of the connections. Specimens of CFS channels with thickness of 1.2mm and 1.5mm in conjunction with various shear distances between the bolts were conducted in order to evaluate the influence of the shear fracture paths and fracture process behaviour on the shear rupture strength of the connections. In order to verify whether existing design equations and methodologies are accurate for determining the shear capacity of the shear beam-end bolted connections under the combined effects of shear-rotation and fracture process behaviour, test results are compared to the current design equations from the AISI S100:16 (R2020) w/S3-22 and AS/NZS 4600:2018. Reliability analyses are carried out to calibrate the capacity reduction/resistance factor for LRFD/LSD.

Finally, the third experimental program involved testing bolted connections under curling effect controlled by monotonic shear loading. Bolt configurations were arranged with relatively long end distances (distance between the outer bolt holes to a free edge) to investigate the curling effect on the fracture behaviour and capacity of the connections. This program focuses on three different failure modes being block shear, tear-out and curling and proposes a set of design equations for block shear and tear-out strengths considering the curling effect which is not available in the current Australian and New Zealand Standard (AS/NZS 4600:2018) and North American Specification (AISI S100:16 (R2020) w/S3-22). A calibrated FEM was subsequently developed to verify against the test results. The reliable data set of the FE models is further used to extend the data range on the basis of parametric study and is finally used for the verification procedure of the proposed formulas over a wider range of data. Reliability analyses are implemented to evaluate the capacity reduction/resistance factors of the proposed models for LRFD/LSD.

STATEMENT OF ORIGINALITY

This is to certify that to the best of my knowledge; the content of this thesis is my own work. This thesis has not been submitted for any degree or other purposes.

I certify that the intellectual content of this thesis is the product of my own work and that all the assistance received in preparing this thesis and sources have been acknowledged.

Viet Binh Pham

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AUTHORSHIP ATTRIBUTION STATEMENT

As co-authors of the following published/submitted journal articles and conference papers, this statement confirms that Mr. Viet Binh Pham's contributions to these publications are consistent with him being named the first author for each, including conceptualization, investigation, data curation, and drafting of manuscripts.

Journal articles

- 1) Pham, V.B., C.H. Pham and G.J. Hancock. 2021. "Block shear behaviour and design of cold-formed staggered hole bolted connections." *Journal of constructional steel research*. 180: 106591. <https://doi.org/10.1016/j.jcsr.2021.106591>.
- 2) Pham, V.B., C.H. Pham and G.J. Hancock. 2022. "New design approach to high-strength steel staggered-hole bolted connections failing in block shear." *Journal of structural engineering*. 148 (5): 04022030. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0003316](https://doi.org/10.1061/(ASCE)ST.1943-541X.0003316).
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CONTENTS

Abstract.....	2
Statement of originality	4
Acknowledgements	5
Authorship attribution statement.....	6
Contents	7
List of figures.....	11
List of tables.....	21
Chapter 1 Introduction.....	24
1.1. Background and problem statements.....	24
1.2. Research objectives and scope	29
1.3. Research methodology	30
1.4. Thesis outline	33
Chapter 2 Literature review	35
2.1. Introduction	35
2.2. Cold-formed steel bolted connections design provisions	35
2.2.1. The Australian/New Zealand Standard AS/NZS 4600:2018.....	35
2.2.1.1 General requirements	35
2.2.1.2 Design shear capacity for bolted connections in shear	36
2.2.1.3 Rupture in bolted connections	38
2.2.1.4 Capacity reduction factors	39
2.2.2. The American Iron and Steel Institute AISI S100-16 Supplement 3 (2022).....	40
2.2.2.1 General requirements	40
2.2.2.2 Design shear capacity for bolted connections in shear	40
2.2.2.3 Rupture in bolted connections	41
2.2.2.4 Resistance factor and safety factor	43
2.2.3. The Eurocode EN 1993-1-8:2005.....	44

2.3. Studies into behaviour of structural bolted connections in shear.....	45
2.4. Studies into rupture strength of bolted connections	46
2.4.1. Rupture in conventional bolted connections	46
2.4.2. Rupture in staggered hole bolted connections	48
2.4.3. Rupture in bolted connections under curling effect.....	49
2.5. Development of dual actuator setup for testing shear beam-end connections	50
2.6. Summary.....	54
Chapter 3 Experimental investigation of staggered hole bolted connections.....	55
3.1. Introduction	55
3.2. Experimental investigation of staggered hole bolted connections	55
3.3. Experimental results	59
3.3.1. Staggered hole bolted connections tests of 2.4mm cold-formed steel sheets.....	59
3.3.2. Staggered hole bolted connections tests of 1.9mm cold-formed steel sheets.....	64
3.3.3. Actual failure path in the inclined shear and tension zone	66
3.3.4. Influence of the combined shear and tension ruptures on the inclined shear and tension zone.....	68
3.4. Design standard and specification comparisons.....	71
3.4.1. Current design equations	71
3.4.2. Proposed design equation	72
3.4.3. Reliability analysis	74
3.5. Summary.....	77
Chapter 4 Behaviour on high strength shear beam-end bolted connections in cold-formed steel channels	79
4.1. Introduction	79
4.2. Experimental investigation of simple shear bolted connections	79
4.3. Experimental results	87
4.3.1. Running ratio $RR=1.0$	87
4.3.1.1 Shear bolted connections tests of 1.2mm cold-formed steel channels.....	87

4.3.1.2 Shear bolted connections tests of 1.5mm cold-formed steel channels.....	94
4.3.2. Running ratios $RR=2.0$ and $RR=3.0$	99
4.4. Design standard and specification comparisons.....	102
4.4.1. Current design equations in standards/specifications.....	102
4.4.2. Effect of the shear distance on the behaviour of the fracture process in the shear zones	104
4.4.3. Comparison of the experimental results with the cold-formed steel design standard in Australia and New Zealand Standard (AS/NZS 4600:2018).....	105
4.4.4. Reliability analysis	109
4.5. Summary	110
Chapter 5 Experimental investigation of bolted connections under curling phenomenon..	112
5.1. Introduction	112
5.2. Experimental investigation of curling effect on bolted connections	112
5.3. Experimental results	116
5.3.1. Bolted connections Configuration Type 1	117
5.3.2. Bolted connections Configuration Type 2.....	120
5.3.3. Bolted connections Configuration Type 3.....	123
5.4. Design standard and specification comparisons.....	127
5.4.1. Current design equations in standards/specifications.....	127
5.4.2. Effect of the curling phenomenon on the stress distribution of the cold-formed plate.....	128
5.4.3. Proposed design equation	130
5.4.4. Reliability analysis	132
5.5. Summary	133
Chapter 6 Finite element modelling of cold-formed steel bolted connections.....	135
6.1. Introduction	135
6.2. Development of the finite element (FE) models.....	135
6.2.1. Selection of ABAQUS solver.....	135
6.2.2. Contacts and interactions.....	135

6.2.3. Element type and mesh size	136
6.2.4. Selection of loading rate	136
6.3. Verification of FE models against the existing experiments	136
6.3.1. Staggered hole bolted connections	136
6.3.1.1 Input parameters.....	136
6.3.1.2 Finite element results	143
6.3.1.3 Parametric study.....	150
6.3.2. Bolted connections under curling effect.....	166
6.3.2.1 Input parameters.....	166
6.3.2.2 Finite element results	168
6.3.2.3 Parametric study.....	174
6.4. Summary	177
Chapter 7 Conclusions.....	179
7.1. Concluding remarks.....	179
7.2. Limitations and suggestions for future research.....	183
Notation list.....	184
References	189
Appendix A Staggered hole bolted connections test results.....	195
Appendix B Coupon test results	197
Appendix C High strength shear beam-end bolted connections in cold-formed steel channels results	199
Appendix D Bolted connections under curling effect test results.....	205
Appendix E FE model results	209

LIST OF FIGURES

Figure 1.1: Cold-formed steel structures in mid-rise residential and commercial buildings (Image courtesy of structuremag.org and framecad.com, respectively)	24
Figure 1.2: (a) Bolted connection from a coped cold-formed steel channel to an I-beam and (b) Autoform® Notched Ends product from Kingspan technical handbook (Image courtesy of kingspan.com).....	25
Figure 1.3: (a) CFS bolted connection with staggered holes and (b) block shear fracture of a specimen observed in a test in this thesis.....	28
Figure 1.4: Potential failure paths of stiffened channel (According to AISI S100:16-C (R2020) w/S3-22 - Section J6 [7])	28
Figure 1.5: (a) CFS bolted connection with relatively long end distance (b) end plate curling of a specimen observed in a test in this thesis.....	28
Figure 1.6: Block shear failure planes (According to AS/NZS 4600:2018 [2])	29
Figure 1.7: (a) Shear beam-end bolted connection geometry and (b) block shear fracture of a specimen observed in a test in this thesis.....	29
Figure 1.8: Schematic of double bolt specimen (Research report RP01-4 – AISI [10])	31
Figure 1.9: Schematic of a shear beam-end bolted connection joining a CFS channel to a tubular column.....	32
Figure 2.1: Types of failures of bolted connections: (a) tear-out failure, (b) bearing failure, (c) net section tension failure and (d) shear fracture in bolt (Yu [16])	36
Figure 2.2: Block shear failure planes (According to AS/NZS 4600:2018 [2])	39
Figure 2.3: Potential failure paths of stiffened channel (According to AISI S100:16-C (R2020) w/S3-22 - Section J6 [7])	43
Figure 2.4: Block shear tearing (According to Eurocode 3 - Design of steel structures - Part 1-8: Design of joints [15])	44
Figure 2.5: Types of failures (Winter [18])	45
Figure 2.6: Dual actuator beam-to-column connection test (Astaneh-Asl et al. [39]).....	50
Figure 2.7: Elevation of test setup using dual actuator (Gong [40]).....	51
Figure 2.8: Beam-to-column shear tab connection test setup (Rogers et al. [43]).....	52

Figure 2.9: Beam-to-column connection test setup (Thomas et al. [44])	52
Figure 2.10: Hybrid test setup for screwed connection from cold-formed channel to RHS beam (Huynh et al. [46]).....	53
Figure 2.11: Hybrid test setup for PAF connection from cold-formed channel to tubular column (Truong et al. [47]).....	53
Figure 3.1: Schematic of (a) test set-up, (b) test specimen, (c) cross-section of test specimen and (d) DIC set-up of a staggered hole bolted connection	57
Figure 3.2: Schematic of (a) staggered hole bolted connections configuration and (b) zones of strain extension	58
Figure 3.3: Schematic of a staggered hole bolted connection (a) after failing in the tension zone and (b) after failing further in the inclined shear and tension zone	58
Figure 3.4: Displacement comparison between DIC and LVDTs at the top edge of the specimen A1.9-08-00	59
Figure 3.5: (a) Test results of 2.4mm cold-formed steel staggered hole bolted connections using M12-Bolts and (b) Von Mises strain distribution of the specimen A2.4-12-45	62
Figure 3.6: Test results of 2.4mm cold-formed steel staggered hole bolted connections using M8-Bolts corresponding to failure process of the specimens A2.4-08-00 and A2.4-08-29	63
Figure 3.7: True strain distribution of the specimens (a) A2.4-12-00 and (b) A2.4-08-00 using DIC.....	63
Figure 3.8: True strain distribution of the specimens (a) A2.4-12-27 and (b) A2.4-08-29 using DIC.....	63
Figure 3.9: True strain distribution of the specimens (a) A2.4-12-45 and (b) A2.4-08-45 using DIC.....	63
Figure 3.10: Test results of 1.9mm cold-formed steel staggered hole bolted connections using M12-Bolts corresponding to failure process of the specimens A1.9-12-00 and A1.9-12-27	65
Figure 3.11: Test results of 1.9mm cold-formed steel staggered hole bolted connections using M8-Bolts corresponding to failure process of the specimen A1.9-08-45	65
Figure 3.12: True strain distribution of the specimens (a) A1.9-12-00 and (b) A1.9-08-00 using DIC.....	65
Figure 3.13: True strain distribution of the specimens (a) A1.9-12-27 and (b) A1.9-08-29 using DIC.....	66

Figure 3.14: True strain distribution of the specimens (a) A1.9-12-45 and (b) A1.9-08-45 using DIC.....	66
Figure 3.15: Von Mises strain distribution of (a) A2.4-12-04 specimen, (b) A2.4-12-10 specimen, (c) A2.4-08-06 specimen and (d) A2.4-08-13 using FEM modelling. Note: details of the specimens are demonstrated in Section 6.3.1.3.	66
Figure 3.16: Actual failure path in a staggered hole bolted connection (a) in general and (b) of the specimen A2.4-12-27.....	67
Figure 3.17: The relationship between the theoretical failure path and measured failure path of the staggered hole bolted connections using (a) M12-Bolts and (b) M8-Bolts.....	67
Figure 3.18: Schematic of staggered hole bolted connections failing in the inclined shear and tension zone of the specimen A2.4-12-00.....	69
Figure 3.19: Schematic of staggered hole bolted connections failing in the inclined shear and tension zone of the specimen A2.4-12-27.....	69
Figure 3.20: Schematic of staggered hole bolted connections failing in (a) the inclined shear and tension zone and (b) the pure tension zone of the specimen A2.4-12-45.	70
Figure 3.21: Shear and tension force at fracture zones.	70
Figure 3.22: Reliability index β with ratio of dead load to live load D/L for different values ...	77
Figure 4.1: Stress-strain curves of (a) 1.2mm G500 cold-formed sheets and (b) 1.5mm G450 cold-formed sheets	81
Figure 4.2: Elevation view of a typical test setup	81
Figure 4.3: Connection details	82
Figure 4.4: Shear buckling behaviour of the CFS channel using the short stiffener (thickness of 6.0mm).....	82
Figure 4.5: Detail of the ideal web stiffener (thickness of 10.0mm)	82
Figure 4.6: Load transfer system	83
Figure 4.7: Lateral restraining system	84
Figure 4.8: Linear variable displacement transducers and inclinometer positions.....	85
Figure 4.9: Schematic of the CFS channels configurations.....	85
Figure 4.10: (a) Test results of 1.2mm cold-formed steel channels using M8-bolts with $e_2 = 15mm$ (b) Fracture process of the specimen T1.2-08-40-1.0-15-01	90

Figure 4.11: Test results of 1.2mm cold-formed steel channels using M8-bolts with $e_2 = 15mm$ (a) shear distance $s = 20mm$, (b) shear distance $s = 30mm$ and (c) shear distance $s = 40mm$	91
Figure 4.12: (a) Test results of 1.2mm cold-formed steel channels using M8-bolts with $e_2 = 20mm$	92
Figure 4.13: Test results of 1.2mm cold-formed steel channels using M8-bolts with $e_2 = 20mm$ (a) shear distance $s = 20mm$, (b) shear distance $s = 30mm$ and (c) shear distance $s = 40mm$	92
Figure 4.14: Test results of 1.2mm cold-formed steel channels using M12-bolts with $e_2 = 20mm$	93
Figure 4.15: Test results of 1.2mm cold-formed steel channels using M12-bolts with $e_2 = 20mm$ (a) shear distance $s = 30mm$, (b) shear distance $s = 40mm$ and (c) shear distance $s = 50mm$	93
Figure 4.16: (a) Test results of 1.5mm cold-formed steel channels using M12-bolts with $e_2 = 20mm$ (b) Fracture process of the specimen T1.5-12-30-1.0-20-02	96
Figure 4.17: Test results of 1.5mm cold-formed steel channels using M12-bolts with $e_2 = 20mm$ (a) shear distance $s = 30mm$, (b) shear distance $s = 40mm$ and (c) shear distance $s = 50mm$	97
Figure 4.18: Test results of 1.5mm cold-formed steel channels using M8-bolts with $e_2 = 15mm$	98
Figure 4.19: Test results of 1.5mm cold-formed steel channels using M8-bolts with $e_2 = 15mm$ (a) shear distance $s = 20mm$, (b) shear distance $s = 30mm$ and (c) shear distance $s = 40mm$	98
Figure 4.20: (a) Load-displacement curve and (b) Shear fracture in bolts of the specimen T1.5-08-40-1.0-20-01 captured on the other side	99
Figure 4.21: Deformation and fracture behaviour of the specimen T1.2-12-40-3.0-25-01 with RR=3.0 captured on the other side.....	101
Figure 4.22: Deformation and fracture behaviour of the specimen T1.2-12-50-2.0-25-02 with RR=2.0	101
Figure 4.23: Comparison of the test results of the specimen T1.2-12-40-RR-25 using different running ratios	102
Figure 4.24: Comparison of the test results of the specimen T1.5-08-30-RR-15 using different running ratios	102
Figure 4.25: Two cases of the block shear fracture process behaviour: (a) Case 1 with the shear distance s relatively short and (b) Case 2 with the shear distance s relatively long	105

Figure 4.26: Comparison of the experimental results with the cold-formed steel design standard AS/NZS 4600:2018 [2] of the 1.2mm cold-formed steel bolted connections (a) using M8-bolts with $e_2 = 15mm$ and (b) using M12-bolts with $e_2 = 20mm$	106
Figure 4.27: Comparison of the experimental results with the cold-formed steel design standard AS/NZS 4600:2018 [2] of the 1.2mm cold-formed steel bolted connections (a) using M8-bolts with $e_2 = 20mm$ and (b) using M12-bolts with $e_2 = 30mm$	106
Figure 4.28: Comparison of the experimental results with the cold-formed steel design standard AS/NZS 4600:2018 [2] of the 1.5mm cold-formed steel bolted connections (a) using M8-bolts with $e_2 = 15mm$ and (b) using M12-bolts with $e_2 = 25mm$	107
Figure 4.29: Comparison of the experimental results with the cold-formed steel design standard AS/NZS 4600:2018 [2] of the 1.2mm cold-formed steel bolted connections (a) using M8-bolts with $e_2 = 20mm$ and (b) using M12-bolts with $e_2 = 30mm$	107
Figure 4.30: Reliability index β with ratio of dead load to live load D/L for different values .	110
Figure 5.1: Test set-up of a bolted connection.....	114
Figure 5.2: Schematic of a typical test specimen of a bolted connection	115
Figure 5.3: Schematic of bolted connections (a) Configuration Type 1, (b) Configuration Type 2 and (c) Configuration Type 3.....	115
Figure 5.4: (a) Test results of 1.0mm cold-formed steel bolted connections Configuration Type 1 using M8-bolts and (b) Fracture process of the specimen T1.0-08-25-1H.....	119
Figure 5.5: Test results of 1.0mm cold-formed steel bolted connections Configuration Type 1 using M12-bolts	120
Figure 5.6: (a) Test results of 1.2mm cold-formed steel bolted connections Configuration Type 2 using M12-bolts and (b) Fracture process of the specimen T1.2-12-30-2H.....	122
Figure 5.7: Test results of 1.2mm cold-formed steel bolted connections Configuration Type 2 using M8-bolts	123
Figure 5.8: Test results of 1.2mm cold-formed steel bolted connections Configuration Type 3 using M12-bolts	125
Figure 5.9: Test results of 1.2mm cold-formed steel bolted connections Configuration Type 3 using M8-bolts	125
Figure 5.10: (a) Test results of 1.9mm cold-formed steel bolted connections Configuration Type 3 using M12-bolts and (b) Fracture process of the specimen T1.9-12-42-4H.....	126

Figure 5.11: Test results of 1.9mm cold-formed steel bolted connections Configuration Type 3 using M8-bolts	127
Figure 5.12: Curling behaviour of specimen (a) T1.2-12-30-2H (FEM) at Stage 4 (i.e., curling occurrence) and (b) T1.0-12-30-1H (FEM) at Stage 4 (i.e., curling occurrence).....	129
Figure 5.13: (a) Stress distribution in the transverse direction (S11) at Stage 3 (i.e., peak load) and (b) the corresponding virtual model of the outstanding piece of plate caused by the bending under the curling phenomenon of the specimen T1.2-12-30-2H (FEM)	129
Figure 5.14: (a) Stress distribution in the transverse direction (S11) at Stage 3 (i.e., peak load) and (b) the corresponding virtual model of the outstanding piece of plate caused by the bending under the curling phenomenon of the specimen T1.0-12-30-1H (FEM)	129
Figure 5.15: Reliability index β with ratio of dead load to live load D/L for different values .	133
Figure 6.1: Finite Element Modelling of the staggered hole bolted connection: (a) the assembly and (b) the bolt in contact with the hole. Note: a load speed of 210 mm/min was applied to the top hot-rolled plate while fixed-end boundary condition was applied to the bottom hot-rolled plate, which was similar to the corresponding experimental program	137
Figure 6.2: (a) Stress-strain curves of 1.9mm cold-formed sheets and (b) stress-strain curves of 2.4mm cold-formed sheets.....	137
Figure 6.3: Stress-strain curves from coupon tests in longitudinal direction and FE input for (a) 1.9mm cold-formed sheets and (b) 2.4mm cold-formed sheets.....	138
Figure 6.4: Schematic of the plane of isotropy at every point in a high strength cold-rolled steel sheet after experiencing a longitudinal rolling process.....	138
Figure 6.5: Comparison of the FE results using transversely isotropic material property and isotropic material property for high strength cold-formed steels against current test results of the specimen A1.9-12-00	139
Figure 6.6: Dependence of the equivalent strain to fracture on the stress triaxiality.....	141
Figure 6.7: (a) Influence of stress triaxiality on the equivalent strain to fracture (Suárez et al. [67]) and (b) various fracture strain formulas	142
Figure 6.8: Comparison of the FE results based on three different calibration curves with the experiments of the 2.4mm cold-formed steel staggered hole bolted connections using M12-Bolts with the angle $\alpha = 0^0$	143

Figure 6.9: Comparison of the FE results with the experiments of the 2.4mm cold-formed steel staggered hole bolted connections using M12-Bolts (a) with the angle $\alpha = 0^0$, (b) with the angle $\alpha = 27^0$ and (c) with the angle $\alpha = 45^0$	145
Figure 6.10: Comparison of the FE results with the experiments of the 2.4mm cold-formed steel staggered hole bolted connections using M8-Bolts (a) with the angle $\alpha = 0^0$, (b) with the angle $\alpha = 29^0$ and (c) with the angle $\alpha = 45^0$	146
Figure 6.11: Comparison of the FE results with the experiments of the 1.9mm cold-formed steel staggered hole bolted connections using M12-Bolts (a) with the angle $\alpha = 0^0$, (b) with the angle $\alpha = 27^0$ and (c) with the angle $\alpha = 45^0$	147
Figure 6.12: Comparison of the FE results with the experiments of the 1.9mm cold-formed steel staggered hole bolted connections using M8-Bolts (a) with the angle $\alpha = 0^0$, (b) with the angle $\alpha = 29^0$ and (c) with the angle $\alpha = 45^0$	148
Figure 6.13: Comparison of failure process of the specimen A2.4-12-45 from FE modelling and from experiments	149
Figure 6.14: Configuration and zones of strain extension of staggered hole bolted connections (a) Configuration Type A, (b) Configuration Type B, (c) Configuration Type C and (d) Configuration Type D.	151
Figure 6.15: FE results of 2.4mm cold-formed steel staggered hole bolted connections Configuration Type A using (a) M12-Bolts and (b) M8-Bolts.....	154
Figure 6.16: FE results of 2.4mm cold-formed steel staggered hole bolted connections Configuration Type B using (a) M12-Bolts and (b) M8-Bolts.	157
Figure 6.17: FE results of 2.4mm cold-formed steel staggered hole bolted connections Configuration Type C using (a) M12-Bolts and (b) M8-Bolts.	159
Figure 6.18: Schematic of failure process of the specimen C2.4-12-65 from FE modelling through 6 stages.....	160
Figure 6.19: Schematic of (a) the specimen D2.4-12-30 and (b) the specimen D2.4-08-30 after failing.	162
Figure 6.20: FE results of 2.4mm cold-formed steel staggered hole bolted connections Configuration Type D using (a) M12-Bolts and (b) M8-Bolts.....	162
Figure 6.21: Schematic of failure process of the specimen D2.4-12-65 from FE modelling through 6 stages.....	163

Figure 6.22: Schematic of typical Finite Element Modelling of the bolted connection (a) Configuration Type 1, (b) Configuration Type 2 and (c) Configuration Type 3. Note: a load speed of 210 mm/min was applied to the top hot-rolled plate while fixed-end boundary condition was applied to the bottom hot-rolled plate, which was similar to the corresponding experimental program.....	167
Figure 6.23: Stress-strain curves from coupon tests in longitudinal direction and FE input for (a) 1.0mm G550 cold-formed sheets, (b) 1.2mm G500 cold-formed sheets, (c) 1.9mm G450 cold-formed sheets and (d) 1.9mm G300 cold-formed sheets	167
Figure 6.24: Comparison of the FE results with the experiments of the 1.0mm G550 cold-formed steel bolted connections Configuration Type 1 using M8-bolts (a) with the end distance $e_1 = 15$, (b) with the end distance $e_1 = 18$ and (c) with the end distance $e_1 = 25$	170
Figure 6.25: Comparison of the FE results with the experiments of the 1.2mm G500 cold-formed steel bolted connections Configuration Type 2 using M12-bolts (a) with the end distance $e_1 = 18$, (b) with the end distance $e_1 = 22$, (c) with the end distance $e_1 = 25$ and (d) with the end distance $e_1 = 30$	171
Figure 6.26: Comparison of the FE results with the experiments of the 1.2mm G500 cold-formed steel bolted connections Configuration Type 3 using M8-bolts (a) with the end distance $e_1 = 17$ and (b) with the end distance $e_1 = 20$	171
Figure 6.27: Comparison of the FE results with the experiments of the 1.9mm G450 cold-formed steel bolted connections Configuration Type 3 using M8-bolts (a) with the end distance $e_1 = 28$, (b) with the end distance $e_1 = 32$, (c) with the end distance $e_1 = 35$ and (d) with the end distance $e_1 = 40$	172
Figure 6.28: Comparison of failure process of the specimen T1.2-12-30-2H between the FEM and the experiment.....	173
Figure 6.29: Comparison of the experimental and calibrated FE results with the proposed design Equation (5.8) in conjunction with the current design formulas from AS/NZS 4600:2018 [2] of the 1.0mm G550 cold-formed steel bolted connections Configuration Type 1 using (a) M8-bolts and (b) M12-bolts	176
Figure 6.30: Comparison of the experimental and calibrated FE results with the proposed design Equation (5.8) in conjunction with the current design formulas from AS/NZS 4600:2018 [2] of	

the 1.2mm G500 cold-formed steel bolted connections Configuration Type 2 using (a) M8-bolts and (b) M12-bolts	176
Figure 6.31: Comparison of the experimental and calibrated FE results with the proposed design Equation (5.8) in conjunction with the current design formulas from AS/NZS 4600:2018 [2] of the 1.2mm G500 cold-formed steel bolted connections Configuration Type 3 using (a) M8-bolts and (b) M12-bolts	177
Figure 6.32: Comparison of the experimental and calibrated FE results with the proposed design Equation (5.8) in conjunction with the current design formulas from AS/NZS 4600:2018 [2] of the 1.9mm (a) G450 cold-formed steel bolted connections Configuration Type 3 using M8-bolts and (b) G300 cold-formed steel bolted connections Configuration Type 3 using M12-bolts	177
Figure A.1: Test results of the cold-formed steel staggered hole bolted connections using M12-Bolts (a) with the angle $\alpha = 0^\circ$, (b) with the angle $\alpha = 27^\circ$ and (c) with the angle $\alpha = 45^\circ$	195
Figure A.2: Test results of the cold-formed steel staggered hole bolted connections using M8-Bolts (a) with the angle $\alpha = 0^\circ$, (b) with the angle $\alpha = 29^\circ$ and (c) with the angle $\alpha = 45^\circ$	196
Figure B.1: Test results of (a) G550 1.0mm steel and (b) G500 1.2mm steel	198
Figure B.2: Test results of (a) G450 1.5mm steel and (b) G450 1.9mm steel	198
Figure B.3: Test results of (a) G300 1.9mm steel and (b) G450 2.4mm steel	198
Figure C.1: Test results of the 1.2mm CFS bolted connections using M12-bolts (RR=1.0) with (a) $e_2 = 20mm$, (b) $e_2 = 25mm$ and (c) $e_2 = 30mm$	199
Figure C.2: Test results of the 1.2mm CFS bolted connections using M8-bolts (RR=1.0) with (a) $e_2 = 15mm$, (b) $e_2 = 20mm$ and (c) $e_2 = 25mm$	200
Figure C.3: Test results of the 1.2mm CFS bolted connections using M12-bolts (RR=2.0) with (a) $e_2 = 25mm$ and (b) $e_2 = 30mm$	201
Figure C.4: Test results of the 1.2mm CFS bolted connections using M8-bolts (RR=2.0) with $e_2 = 20mm$	201
Figure C.5: Test results of the 1.5mm CFS bolted connections using M12-bolts (RR=1.0) with (a) $e_2 = 20mm$, (b) $e_2 = 25mm$ and (c) $e_2 = 30mm$	202
Figure C.6: Test results of the 1.5mm CFS bolted connections using M8-bolts (RR=1.0) with (a) $e_2 = 15mm$, (b) $e_2 = 20mm$ and (c) $e_2 = 25mm$	203

Figure C.7: Test results of the 1.5mm CFS bolted connections using M12-bolts (RR=2.0) with $e_2 = 25mm$	204
Figure C.8: Test results of the 1.5mm CFS bolted connections using M8-bolts (RR=1.0) with $e_2 = 15mm$	204
Figure D.1: Test results of the 1.0 mm G550 cold-formed steel bolted connections Configuration Type 1 using (a) M8-bolts and (b) M12-bolts	205
Figure D.2: Test results of the 1.2 mm G500 cold-formed steel bolted connections Configuration Type 2 using (a) M8-bolts and (b) M12-bolts	206
Figure D.3: Test results of the 1.2 mm G500 cold-formed steel bolted connections Configuration Type 3 using (a) M8-bolts and (b) M12-bolts	207
Figure D.4: Test results of the 1.9 mm (a) G450 cold-formed steel bolted connections Configuration Type 3 using M8-bolts and (b) G300 cold-formed steel bolted connections Configuration Type 3 using M12-bolts.....	208
Figure E.1: FE results of 1.9mm cold-formed steel staggered hole bolted connections Configuration Type A using (a) M12-Bolts and (b) M8-Bolts.....	209
Figure E.2: FE results of 1.9mm cold-formed steel staggered hole bolted connections Configuration Type B using (a) M12-Bolts and (b) M8-Bolts.	209
Figure E.3: FE results of 1.9mm cold-formed steel staggered hole bolted connections Configuration Type C using (a) M12-Bolts and (b) M8-Bolts.	210
Figure E.4: FE results of 1.9mm cold-formed steel staggered hole bolted connections Configuration Type D using (a) M12-Bolts and (b) M8-Bolts.....	210

LIST OF TABLES

Table 2.1: Minimum size of bolt holes (AS/NZS 4600:2018 [2]).....	36
Table 2.2: Bearing factor (C) (AS/NZS 4600:2018 [2])	37
Table 2.3: Modification factor (m_f) for type of bearing connection (AS/NZS 4600:2018 [2])	37
Table 2.4: Capacity reduction factor (ϕ) of bolted connections (AS/NZS 4600:2018 [2])	39
Table 2.5: Minimum size of bolt holes (AISI S100:16 (R2020) w/S3-22 [1]).....	40
Table 2.6: Bearing factor (C) (AISI S100:16 (R2020) w/S3-22 [1])	40
Table 2.7: Resistance factor (ϕ) and safety factor (Ω) of bolted connections (AISI S100:16 (R2020) w/S3-22 [1]).....	43
Table 3.1: Average material properties of steel sheets (full test data in Appendix B)	56
Table 3.2: Specimen configurations	58
Table 3.3: Results of staggered holes bolted connections of 2.4mm cold-formed sheets	60
Table 3.4: Results of staggered holes bolted connections of 1.9mm cold-formed sheets	64
Table 3.5: Comparison of the proposed design Equation (3.2) with the test results and the design standards and specifications from AISI S100:16 (R2020) w/S3-22 [1] and AS/NZS 4600:2018 [2]	73
Table 3.6: Comparison of the proposed design equation with the independent test results on unconventional staggered block shear failures in Teh and Yazici [36] and the specifications from AISI S100:16 (R2020) w/S3-22 [1].....	74
Table 3.7: Resistance factor ϕ and safety factor Ω of the proposed formula with $D/L = 0.2$ and $\beta = 3.5$	77
Table 4.1: Average material properties of the CFS channels (full test data in Appendix B)	80
Table 4.2: Specimen configurations	86
Table 4.3: Test results of the 1.2mm cold-formed steel shear bolted connections with running ratio $RR=1.0$	89
Table 4.4: Test results of the 1.5mm cold-formed steel shear bolted connections with running ratio $RR=1.0$	95

Table 4.5: Test results of the cold-formed steel shear bolted connections with running ratios RR=2.0 and RR=3.0.....	100
Table 4.6: Comparison of the experimental results with the cold-formed steel design standard in Australia and New Zealand Standard (AS/NZS 4600:2018 [2]) for the cold-formed steel bolted connections	108
Table 4.7: Resistance factor ϕ for $D/L = 0.2$ and $\beta = 3.5$	110
Table 5.1: Material properties of steel sheets (full test data in Appendix B)	113
Table 5.2: Specimen configurations	116
Table 5.3: Results of bolted connections Configuration Type 1	117
Table 5.4: Results of bolted connections Configuration Type 2	121
Table 5.5: Results of bolted connections Configuration Type 3	124
Table 5.6: Comparison of the experimental results with the proposed design Equation (5.8) for the cold-formed steel bolted connections	131
Table 5.7: Resistance factor ϕ and safety factor Ω of the proposed formula with $D/L = 0.2$ and $\beta = 3.5$	133
Table 6.1: Material properties of cold-formed sheet steels in the FE simulation extracted from the coupon tests in the longitudinal and transverse directions of the staggered hole bolted connections	139
Table 6.2: Comparison of the FE results using transversely isotropic material property and isotropic material property for high strength cold-formed steels against current test results of the specimen A1.9-12-00	139
Table 6.3: Quantitate values of different calibration curves for the ductile fracture model for high strength cold-formed steel.....	142
Table 6.4: Comparison of the FE results with the experimental results of staggered holes bolted connections of cold-formed sheets of thickness of 1.9mm and 2.4mm.....	144
Table 6.5: FE results of staggered holes bolted connections Configuration Type A of cold-formed sheets of thickness of 1.9mm and 2.4mm	153
Table 6.6: FE results of staggered holes bolted connections Configuration Type B of cold-formed sheets of thickness of 1.9mm and 2.4mm	156
Table 6.7: FE results of staggered holes bolted connections Configuration Type C of cold-formed sheets of thickness of 1.9mm and 2.4mm	158

Table 6.8: FE results of staggered holes bolted connections Configuration Type D of cold-formed sheets of thickness of 1.9mm and 2.4mm	161
Table 6.9: Comparison of the proposed design Equation (3.2) with the calibrated FE results and the cold-formed steel design standards AISI S100:2016 (R2020) w/S3-22 [1] and AS/NZS 4600:2018 [2].....	164
Table 6.10: Comparison of the FE results with the experimental results of the bolted connections	169
Table 6.11: Comparison of the calibrated FE results with the proposed design Equation (5.8) for the cold-formed steel bolted connections	174
Table B.1: Test results of the cold-formed sheet steels under tensile load.....	197

CHAPTER 1

INTRODUCTION

This chapter begins with outlining the motivation of the present research and its significance. Then, results from an extensive literature survey are briefly presented in Chapter 2 not only to identify the existing gaps of knowledge but also to support the novel aspects of the proposed work. A clear set of objectives, scope of work, research procedure and essential methodology are, subsequently summarised to demonstrate the solid framework of the present research. Finally, outcomes and contributions to related communities are addressed.

1.1. BACKGROUND AND PROBLEM STATEMENTS

In recent years, cold-formed steel (CFS) structural members have been widely utilised in mid-rise residential and commercial buildings by means of their light weight and high strength steel properties (see Figure 1.1). CFS members are typically made from thin-gauge steel which makes them lighter than traditional hot-rolled steel (HRS) members. Therefore, using lightweight materials can help builders easily handle and transport them and can also reduce the overall weight of a building, which can have benefits for seismic and wind load resistance. Despite these advantages, CFS members are also quite strong thanks to the inherent strength of steel so that they can be used to support significant loads and can be designed to meet a range of performance requirements. Furthermore, CFS members are often fabricated off-site to reduce construction timelines and minimize waste and scrap.

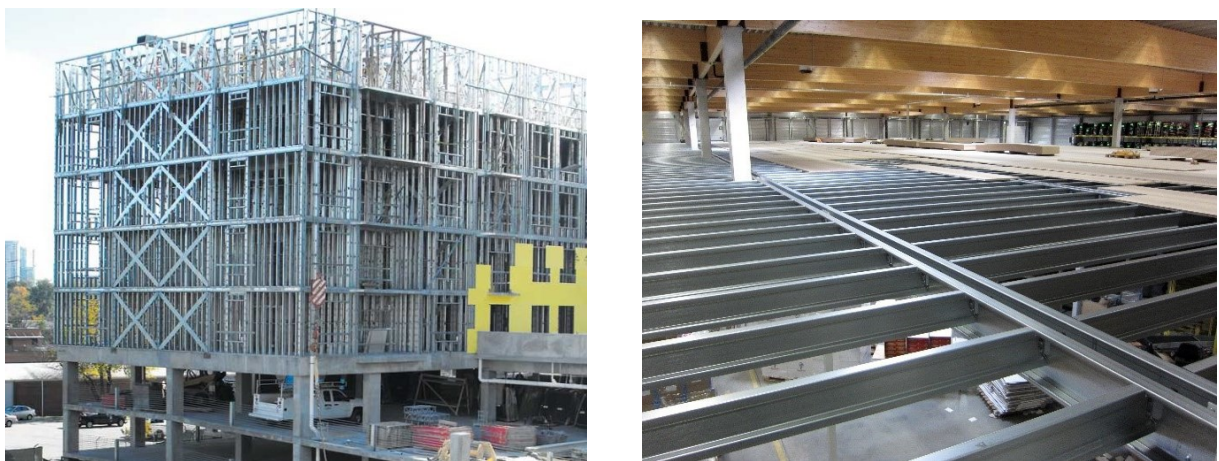
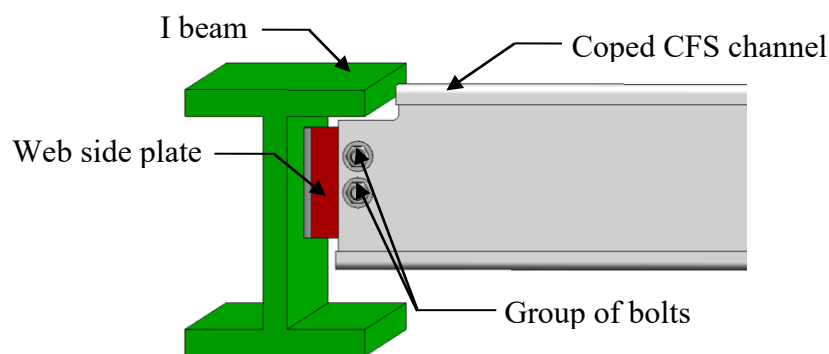


Figure 1.1: Cold-formed steel structures in mid-rise residential and commercial buildings (Image courtesy of structuremag.org and framecad.com, respectively)

For residential and commercial buildings, CFS members are connected to CFS members or HRS members using a variety of methods including welding, bolting, screwing or Power-

Actuated Fasteners (PAFs). However, bolted connections can be commonly used in comparison with other types of connectors such as screws or PAFs due to the high strength resistance and reliability. The bolts are typically inserted through holes in the steel members and tightened using nuts and washers. This creates a secure and rigid connection that can withstand significant loads. One advantage of bolted connections is that they are relatively easy to install and remove, making them a versatile choice for construction projects whereas screwed connections may be more suitable for lighter loads and non-critical applications, such as attaching cladding or trim to a building.

Bolted connections are often designed with a group of bolts in order to increase the shear strength of the connections (Figure 1.2a & b). The bolts are typically arranged in a pattern that distributes the load evenly across the connection, which helps to prevent any one bolt from bearing an excessive load. As studied, the fracture behaviour (i.e., block shear and tear-out failure modes are common failure modes that can occur in bolted connections) and capacity of a group of bolts can be significantly affected by different configurations of bolt arrangements and loading conditions.



(a)



(b)

Figure 1.2: (a) Bolted connection from a coped cold-formed steel channel to an I-beam and (b) Autoform® Notched Ends product from Kingspan technical handbook (Image courtesy of kingspan.com)

Consequently, the main purpose of the research work is to investigate the fracture behaviour of CFS bolted connections in different configurations of bolt arrangements (e.g., staggered hole bolted connections in Chapter 3 and relatively long end distance between the outer bolt holes to a free edge in Chapter 5) and loading conditions (e.g., combined effects of shear and rotation in Chapter 4), which were then conducted based on the concepts, techniques, design guidelines and criteria in the Research Methodology Section. Possible failure modes in bolted connections are bearing, bolt fracture, tear-out, net section tension, shear rupture, tension rupture and block shear rupture as pointed out in AISI S100:16 (R2020) w/S3-22 [1] and AS/NZS 4600:2018 [2]. However, the major failure mode in all Chapters 3, 4 and 5 is the block shear rupture since it initiates the failure with the lowest capacity. Although each chapter separately investigates each problem, all of the failed connections involve fracture (block shear rupture) of the CFS members which is the main purpose of the research work. Hence, the shear strength in the pure shear zones and the tensile strength in the pure tensile zones used in Chapters 3, 4 and 5 are similar. Nevertheless, the combined shear and tensile strengths in the staggered zones, the combined effects of shear and rotation on the connection strength and the effects of curling on the block shear strength are respectively demonstrated in Chapters 3, 4 and 5 as the key findings of the particular chapters.

Firstly, in practical applications, groups of bolts may be assembled with staggered holes as shown in Figure 1.3a. Staggered holes refer to a pattern of bolt holes that are arranged in an offset pattern, rather than being aligned in a straight line. Staggered hole patterns are often used in structures since it is necessary to distribute loads evenly across multiple bolts, such as in flange joints, pressure vessels, and other types of mechanical connections. A method for determining the block shear strength of the staggered hole bolted connections in AISI S100:16 (R2020) w/S3-22 [1] has been employed on the basis of Cochrane's formula (See Figure 1.4). Nevertheless, this methodology may not be accurate for the staggered paths by treating them as tension paths through the stagger and gauge terms since the stagger plane is close to the line of the force. In this case, the force on the stagger plane is mainly in shear and not tension. Likewise, AS/NZS 4600:2018 [2] does not include specific equations for staggered block shear failures. Nevertheless, this special configuration may provide different fracture behaviour in the staggered paths as shown in Figure 1.3b, rather than illustrating pure shear or pure tension ruptures in the conventional configurations. The staggered failure paths in Figure 1.3a are subjected to a combination of shear and tension. Design models for this situation need to be developed and validated.

Secondly, the end distance between the outer bolt holes to a free edge may be relatively long as shown in Figure 1.5a so that a curling phenomenon may occur as shown in Figure 1.5b. In

bolted connections, curling is a type of buckling that can occur in thin steel plates or sections when they are subjected to bending moment, axial or shear forces, which can change the fracture behaviour and lead to a loss of strength and stiffness in the connection (Kim et al. [3] and Kim and Kuwamura [4]). However, design models for this situation have not been properly studied without considering the curling effect or only adding a coefficient accounting for the curling effect into the original block shear capacity. Design models for curling need to be developed.

Thirdly, although the behaviour and strength of HRS shear beam-end connections under different loading conditions have been studied thoroughly, there is very little specific research on the fracture behaviour and capacity of CFS bolted connections in the literature. For instance, Hogan and Munter [5] provided design procedures in the Australian Steel Institute Design Guides for HRS members which consisted of design checks for shear, bending, block shear and failure of single and double angle cleat connectors. Likewise, testing beam-to-column shear connections using two actuators to control the shear force and rotation of the beam was performed by Astaneh-Asl et al. [6] to investigate the behaviour of shear connections for HRS beams. Similar research for cold-formed connections appears to be non-existent while the design guides have only been established for hot-rolled structural members. Despite the fact that the behaviour characteristics of the two types of connections are different because of their different material properties and higher slenderness. On the contrary, existing design equations and methodologies for block shear strength of the CFS bolted connections in AISI S100:16 (R2020) w/S3-22 [1] and AS/NZS 4600:2018 [2] have been only developed based on simply symmetric connections with which only transfer shear and have a negligible resistance to rotation (See Figure 1.6). Consequently, testing shear beam-end bolted connection for CFS channel (Figure 1.7a) is performed in this thesis to investigate the fracture behaviour and capacity of the shear connections (asymmetric connections) under combined effects of shear and rotation (loading conditions) where the applications of this type of connection is commonly used for the primary steel frames or the mezzanine floor systems as shown in Figure 1.1b. A beam-end connection with fracture planes is shown in Figure 1.7b.

Therefore, in this thesis, it is of interest to investigate these three different problems to provide rational design procedures for CFS bolted connections which may include bolting with staggered holes, bolted connections including curling effects and shear beam-end connections under combined effects of shear-rotation. All of the failed connections involve fracture of the CFS members so that fracture behaviour is an important part of this thesis. Last but not least, the research is carried out in a quite common framework which allows to provide guidelines to analyse various scenarios applicable to practical applications.

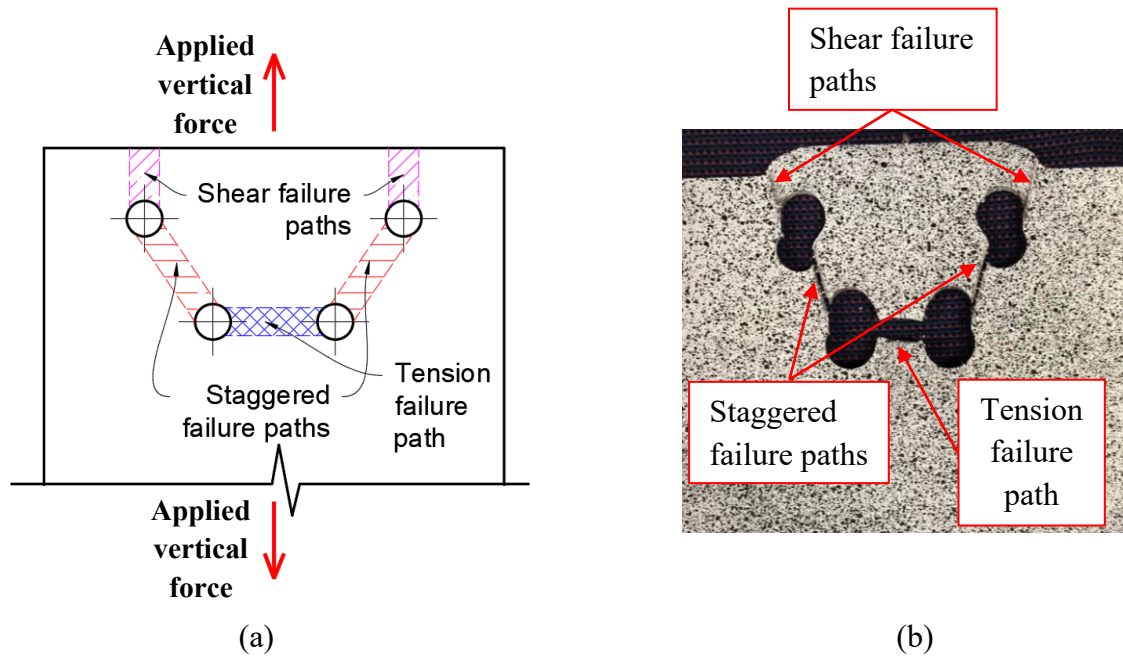


Figure 1.3: (a) CFS bolted connection with staggered holes and (b) block shear fracture of a specimen observed in a test in this thesis

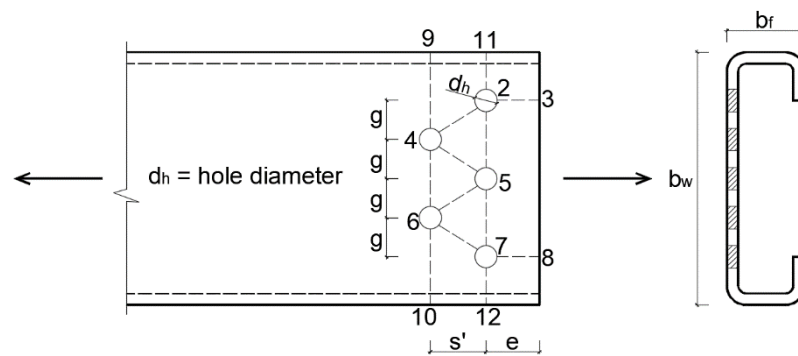


Figure 1.4: Potential failure paths of stiffened channel (According to AISI S100:16-C (R2020) w/S3-22 - Section J6 [7])

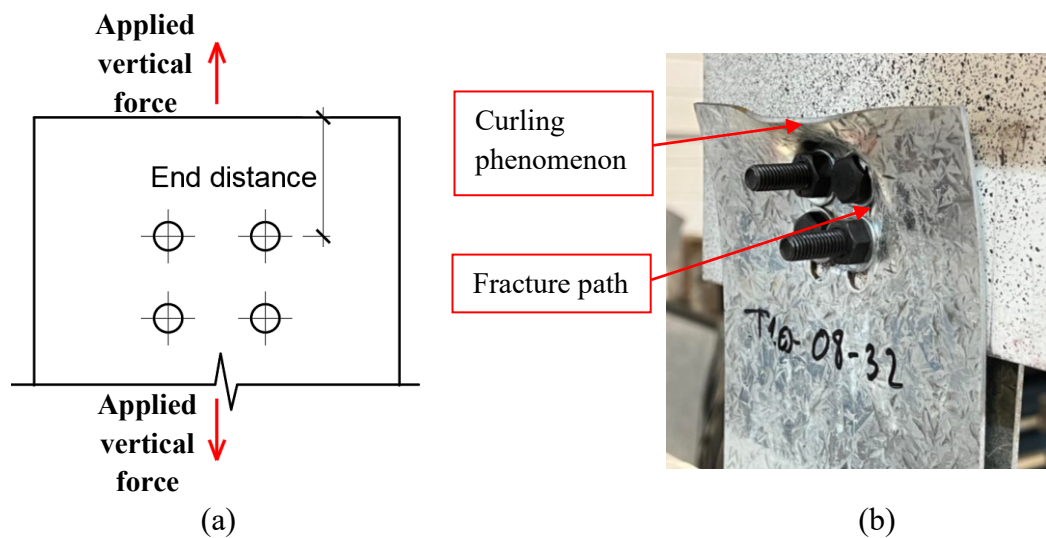


Figure 1.5: (a) CFS bolted connection with relatively long end distance (b) end plate curling of a specimen observed in a test in this thesis

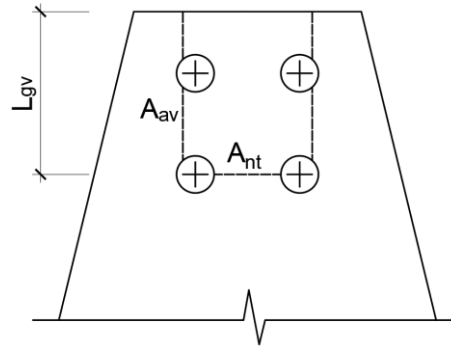


Figure 1.6: Block shear failure planes (According to AS/NZS 4600:2018 [2])

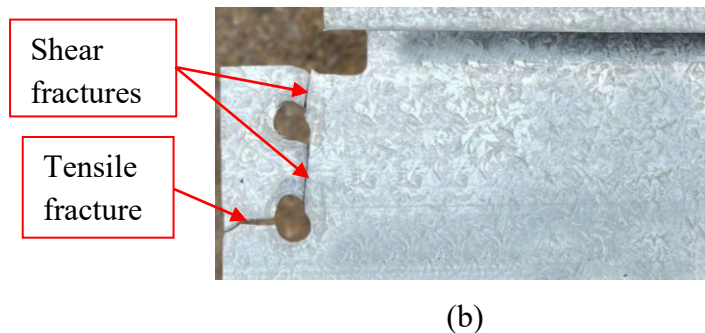
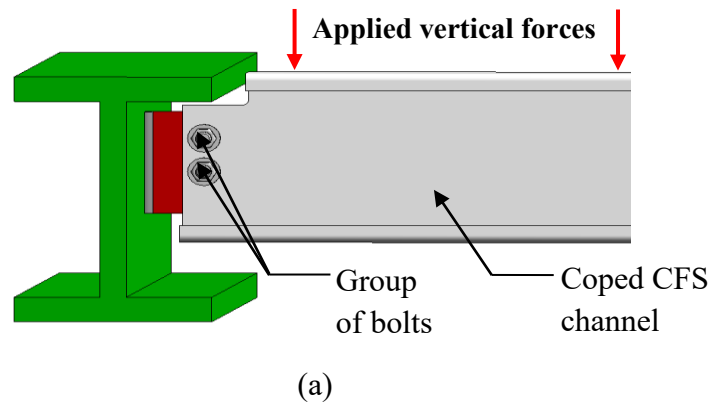


Figure 1.7: (a) Shear beam-end bolted connection geometry and (b) block shear fracture of a specimen observed in a test in this thesis

1.2. RESEARCH OBJECTIVES AND SCOPE

The research described in this thesis aims to provide a better understanding of the fracture behaviour of cold-formed steel bolted connections failing in the block shear or tear-out failure modes under different configurations and loading conditions. The following components have been investigated to achieve the outcomes:

1. An experimental investigation of staggered hole bolted connections is performed to understand the fracture behaviour and block shear strength of the connections with staggered holes. Test results are used to propose a design equation for block shear strength accounting for the combined actions of shear and tension ruptures in the staggered zones. Reliability

analyses are performed to calibrate the capacity reduction/resistance factor for Limit State Design (LSD)/Load and Resistance Factor Design (LRFD).

2. An experimental program of shear beam-end bolted connections joining CFS channels to a tubular column is conducted to investigate the fracture behaviour and block shear strength of the shear beam-end connections. The test apparatus is designed to impose different combinations of shear and rotation which reflects the actual loading conditions in simple connection. Test results are used to validate existing design equations and methodologies for determining the shear capacity of shear beam-end bolted connections subjected to the combined effects of shear-rotation.
3. An experimental program of bolted connections in thin sheet steels with relatively long end distances to investigate the fracture behaviour and shear strength of the connections under curling phenomenon which can occur with thin sheet steels. Test results are used to propose a design equation for block shear and tear-out strengths accounting for the curling effect.
4. Development of Finite Element (FE) models to simulate the strength, fracture behaviour and ductility of bolted connections. The models are verified and calibrated to guarantee the accuracy, robustness and reliability for parametric studies.

1.3. RESEARCH METHODOLOGY

Three series of the experimental programs were conducted in the J.W. Roderick Laboratory for Materials and Structures at the University of Sydney: (i) testing of the staggered hole bolted connections failing in block shear, (ii) testing of the shear beam-end bolted connections joining CFS channels to a tubular column using dual actuator and (iii) testing of the bolted connections failing in block shear and tear-out under curling effect. Testing methodologies are employed by following the AISI S905-13: Test Standard for Cold-formed Steel Connections [8], Specification for Structural Joints Using ASTM A325 or A490 Bolts [9], Research report RP01-4 – AISI: Testing of Bolted Cold-Formed Steel Connections in Bearing (With and Without Washers) [10], and the European Recommendations – ECCS TC7 TWG 7.10: The Testing of Connections with Mechanical Fasteners in Steel Sheeting and Sections [11].

The first test apparatus is designed in accordance with the Research report RP01-4 – AISI [10] for double shear which is shown in Figure 1.8. In particular, the bolts are arranged in a staggered manner as demonstrated in Figure 1.3. The full field of the displacement and the block shear failure mode of the staggered hole bolted connections are captured by means of Digital Image Correlation (DIC) [12] to investigate the actual fracture lengths between the bolts in the staggered zones. In addition, a scanning electron microscope (SEM) technique is used to observe

the experimented fracture behaviour in the fracture zones. This technique is capable of evaluating the effect of the combination of shear and tension ruptures in the staggered zones of the connections. The effect of the staggered paths on the fracture behaviour and capacity of the connections can be investigated by varying distances and angles between staggered holes.

For the second test apparatus in this thesis, a method for testing shear beam-end bolted connections from cold-formed channels to a tubular column using two actuators is demonstrated in Figure 1.9. The shear connection type is made from a T-shaped gusset that is bolted to a tubular column and bolted to a CFS channel. The vertical forces are applied through two actuators. In particular, one actuator (Actuator B) which is close to the connection is used to apply the vertical shear, while the remaining actuator (Actuator A) provides beam end rotation. Two actuators can run at different speeds that will create rotation and moment at the connections. Therefore, various combinations of shear and rotation can be transferred into the connections in order to investigate the fracture behaviour and shear capacity of the connections under different loading conditions. This experimental program focuses on the block shear failure mode with the studied parameters including the shear distances between holes, the bolt diameters, the thicknesses and strength of the CFS materials, the ratio of speeds between two actuators.

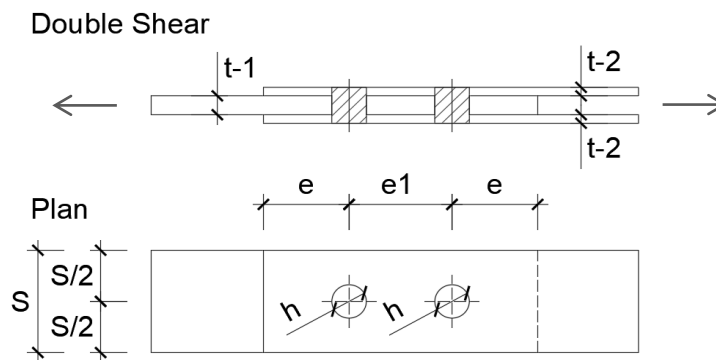


Figure 1.8: Schematic of double bolt specimen (Research report RP01-4 – AISI [10])

The experimental program in the third test is established based on the methodologies in the first test (i.e., using double shear testing apparatus). In particular, the bolts are arranged in a manner as demonstrated in Figure 1.5. The thin thicknesses of the cold-formed sheet steels in conjunction with various relatively long end distances are carried out to evaluate the effect of the curling phenomenon on the fracture behaviour and shear strength of the bolted connections. The vertical and horizontal displacements of the cold-formed sheet steels are recorded by a data logger through the linear variable displacement transducers (LVDTs).

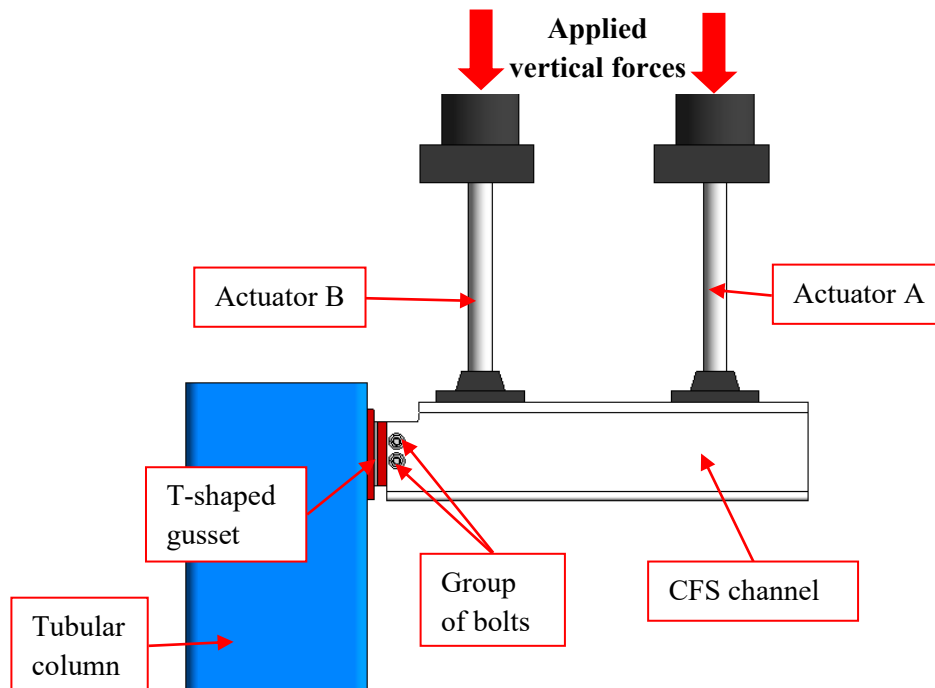


Figure 1.9: Schematic of a shear beam-end bolted connection joining a CFS channel to a tubular column

In each experiment, the test results are processed and used for the analytical studies which include (i) proposing a design formula for block shear strength accounting for the combined actions of shear and tension ruptures in the staggered zones (first test), (ii) validating the design equations of the standards and specifications for block shear strength of the shear beam-end bolted connections (second test) and (iii) proposing design formulas for block shear and tear-out strengths considering the curling effect (third test). Reliability analyses are implemented in accordance with AISI S100:16 (R2020) w/S3-22 [1] and AS/NZS 4600:2018 [2] to determine the capacity reduction/resistance factor for LSD/LRFD. The properties of the steel material used in the experiments are extracted from coupon tests in accordance with AS 1391:2007 [13].

In conjunction with the experimental and analytical studies, numerical studies are also performed. Finite Element Model (FEM) procedure involving numerical simulation of three above series of tests is established using the ABAQUS software [14]. Calibration using the tensile coupon tests is required to determine the necessary input parameters for the material properties, fracture and damage models. In the simulation, the bolt model uses solid deformable elements to match the real behaviour of the bolts. Furthermore, to simulate the plasticity phase for the transversely isotropic material behaviour (i.e., the behaviour of the steel sheets after experiencing a longitudinal rolling process), a Hill's plasticity model is employed using the coupon test results in the longitudinal direction as the reference data. In addition, a "Ductile Damage" model is utilised for the fracture model in the ABAQUS simulation. Required parameters in this model include the fracture locus of the fracture strain versus the stress triaxiality and the strain rate. The

FE models are verified against corresponding test results so that reasonable agreement is achieved for both the strength and failure modes.

1.4. THESIS OUTLINE

- Chapter 1 demonstrates the problem statement on cold-formed steel bolted connections in conjunction with objectives, research methodology and contribution.
- Chapter 2 includes a comprehensive review on the relevant literature with three major fields of structural behaviour of bolted connections, FEM and structural experiments of bolted connections.
- Chapter 3 provides the experimental investigation of staggered hole bolted connections in shear. The test results are discussed and compared with current design standards and specifications to propose a proper design formula. A verification procedure of the proposed formula is carried out based on the reliable data sets from the test and other researchers. Reliability analyses are conducted to evaluate the capacity reduction/resistance factors of the proposed model.
- Chapter 4 describes the experimental investigation of the high strength shear beam-end bolted connections using the dual actuator test-rig. The test program is carried out involving block shear fracture of the bolted connections based on various combined effects of shear forces and rotations. Experimental results are discussed and compared with current design standards and specifications in order to validate whether existing design equations and methodologies are accurate for determining the shear capacity of the shear beam-end bolted connections under the combined effects of shear-rotation and fracture process behaviour. Reliability analyses are also performed to calibrate the capacity reduction/resistance factors for design.
- Chapter 5 describes the experimental program of the thin sheet steels bolted connections under curling effect. A comparison between the test results and current design standards in conjunction with other data sets is discussed in this chapter in order to provide a set of revised design equations. Reliability analyses are employed to evaluate the capacity reduction/resistance factors of the proposed model.
- Chapter 6 illustrates the development of the FEM validated against the previous tests. Critical aspects of the model, such as selection of ABAQUS solver, contacts and interactions between surfaces, the fracture criteria of the materials, loading rate, element type and mesh sensitivity, are calibrated and thoroughly discussed to provide accurate and robust FE models. The model is further used to perform parametric studies for a wide range of the corresponding experiments and to validate the proposed design formulations.

- Chapter 7 summarises significance of this research and provides recommendations for further investigations in future research.

CHAPTER 2

LITERATURE REVIEW

2.1. INTRODUCTION

This chapter primarily reports results from the review of the literature relevant to the present study and design philosophy from current standards and specifications, which not only displays the historical development but also identifies the key knowledge gaps

This chapter firstly introduces the design philosophy of the bolted connections for CFS structures conforming to the Joint Australian/New Zealand Standard – Cold-formed steel structures AS/NZS 4600:2018 [2], the American Iron and Steel Institute AISI S100-16 Supplement 3 (AISI S100:16 (R2020) w/S3-22 [1]), and the Eurocode 3 - Design of steel structures - Part 1-8: Design of joints [15].

Secondly, the chapter focuses on the relevant experimental and numerical studies into the behaviour and rupture strength of bolted connections. Lastly, the development of the dual actuator test methodology and its applications in testing of shear beam-end bolted connections are discussed.

2.2. COLD-FORMED STEEL BOLTED CONNECTIONS DESIGN PROVISIONS

2.2.1. The Australian/New Zealand Standard AS/NZS 4600:2018

2.2.1.1 General requirements

The Australian/New Zealand design provisions shall apply to bolted connections for CFS structural members in which the thickness of a connected part is less than 3mm. In addition, the design provisions recommend that the minimum distance between centres of bolt holes shall provide clearance for bolt heads, nuts, washers and the wrench but shall be not less than 3 times the nominal bolt diameter (d_f). Also, the distance from the centre of any standard hole to the end or other boundary of the connecting member shall be not less than $1.5 d_f$. The minimum size of bolt holes specified is summarised in Table 2.1.

Table 2.1: Minimum size of bolt holes (AS/NZS 4600:2018 [2])

Nominal bolt diameter d_f (mm)	Standard hole diameter d_h (mm)	Oversized hole diameter d_h (mm)	Short-slotted hole dimensions (mm)	Long-slotted hole dimensions (mm)
< 12	$d_f + 1.0$	$d_f + 2.0$	$(d_f + 1.0)$ by $(d_f + 6.0)$	$(d_f + 1.0)$ by $2.5d_f$
≥ 12	$d_f + 2.0$	$d_f + 3.0$	$(d_f + 2.0)$ by $(d_f + 6.0)$	$(d_f + 2.0)$ by $2.5d_f$

2.2.1.2 Design shear capacity for bolted connections in shear

For bolted connections in shear, the design shear capacity is the minimum of the capacities calculated from four limit states including tear-out, net section tension, bearing, and bolts in shear (Figure 2.1).

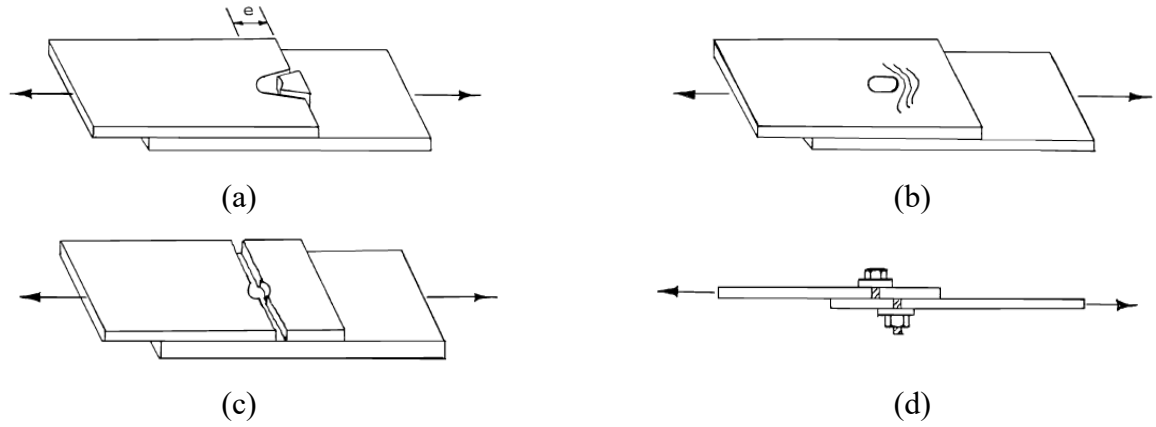


Figure 2.1: Types of failures of bolted connections: (a) tear-out failure, (b) bearing failure, (c) net section tension failure and (d) shear fracture in bolt (Yu [16])

The tear-out strength of the bolted connections without the curling effect is specified as follows

$$V_f = te_1 f_u \quad (2.1)$$

where V_f denotes the tear-out strength of the connections; t is the thickness of the connected part; e_1 is the distance measured in the line of force from the centre of a standard hole to the nearest edge of an adjacent hole or to the end of the connected part; and f_u is the tensile strength of the connected parts.

The design tensile capacity (N_f) on the net section of the connected part is determined as follows

$$N_f = (0.9 + 0.1d_f/s_f) A_{nt} f_u \quad (2.2)$$

where d_f denotes the bolt diameter; s_f denotes the bolt spacing in the tensile resistance plane; and A_{nt} is the net area of the connected part.

The bearing capacity (V_b) without considering bolt hole deformation is determined as follows

$$V_b = m_f C d_f t f_u \quad (2.3)$$

where C is the bearing factor given in Table 2.2; and m_f is the modification factor for the type of bearing connection given in Table 2.3.

Table 2.2: Bearing factor (C) (AS/NZS 4600:2018 [2])

Thickness of connected part t (mm)	Ratio of fastener diameter to member thickness (d_f/t)	C
$0.42 \leq t < 4.76$	$d_f/t < 10$	3.0
	$10 \leq d_f/t \leq 22$	$4 - 0.1(d_f/t)$
	$d_f/t > 22$	1.8

Table 2.3: Modification factor (m_f) for type of bearing connection (AS/NZS 4600:2018 [2])

Type of bearing	m_f
Single shear and outside sheets of double shear connection with washers under both bolt head and nut	1.00
Single shear and outside sheets of double shear connection without washers under both head and nut, or with only one washer	0.75
Single shear and outside sheets of double shear connection using oversized or short-slotted holes parallel to the applied load without washers under both bolt head and nut, or with only one washer	0.70
Single shear and outside sheets of double shear connection using short-slotted holes perpendicular to the applied load without washers under both bolt head and nut, or with only one washer	0.55
Inside sheet of double shear connection with or without washers	1.33
Inside sheet of double shear connection using oversized or short-slotted holes parallel to the applied load with or without washers	1.10
Inside sheet of double shear connection using short-slotted holes perpendicular to the applied load with or without washers	0.90

The bearing capacity (V_b) when deformation around a bolt hole being a design consideration is given by

$$V_b = (0.183t + 1.53) d_f t f_u \quad (2.4)$$

The nominal shear capacity (V_{fv}) of a bolt is determined by

$$V_{fv} = 0.62f_{uf}(n_n A_c + n_x A_o) \quad (2.5)$$

where f_{uf} is the minimum tensile strength of the bolt; n_n denotes the number of shear planes with threads intercepting the shear plane; A_c is the minor diameter area of the bolt; n_x denotes the number of shear planes without threads intercepting the shear plane; and A_o is the plain shank area of the bolt.

2.2.1.3 Rupture in bolted connections

At beam-end connections, where one or more flanges are coped and failure may occur along a plane through the fasteners, the nominal shear rupture capacity (R_{nv}) of a beam-end connection is determined as follows

$$R_{nv} = 0.6f_u A_{nv} \quad (2.6)$$

where A_{nv} is the net area of the web ($A_{nv} = (h_{wc} - n_b d_h)t$); h_{wc} is the coped depth of the web; n_b is the number of holes in the critical plane; and d_h is the diameter of the hole.

The nominal tension rupture capacity (R_{nt}) along a path in affected elements of connected members is determined as follows

$$R_{nt} = 0.85k_t f_u A_{nt} \quad (2.7)$$

where k_t is the correction factor for distribution of forces ($k_t = 1.0$).

The nominal block shear rupture capacity of a beam-end or tension member connection (R_n) shall be determined as follows

$$R_n = 0.6f_y A_{av} + f_u A_{nt} (0.9 + 0.1d_f/s_f) \quad (2.8)$$

in which A_{av} is the active shear area lying in the middle of the net and gross shear areas ($A_{av} = 2L_{av}t$); $L_{av} = L_{gv} - n_r d_h / 2$; L_{gv} is the distance from free edge to centreline of bolt furthest from the edge (Figure 2.2); n_r is the number of rows of bolts; the variable d_f denotes the bolt diameter; and s_f denotes the bolt spacing in the tensile resistance plane.

The design Equation (2.8) does not include explicit equations for either staggered hole block shear failure or bolted connections under the curling effect where the plate toward the edge bends out of plane as described in Chapter 1.

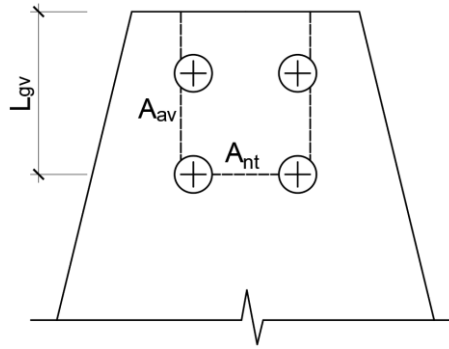


Figure 2.2: Block shear failure planes (According to AS/NZS 4600:2018 [2])

2.2.1.4 Capacity reduction factors

For Limit State Design (LSD), AS/NZS 4600:2018 [2] uses the Load Resistance Factor Design (LRFD) approach to account for the uncertainties in design, fabrication, and the estimation of applied loads. The capacity reduction factor (ϕ) is used to account for the uncertainties and variabilities as given in Equation (2.9). The values of the capacity reduction factors are listed in Table 2.4.

$$R_d = \phi R_u \quad (2.9)$$

in which R_d is the design capacity; and R_u denotes the nominal capacity.

Table 2.4: Capacity reduction factor (ϕ) of bolted connections (AS/NZS 4600:2018 [2])

Design capacity	Capacity reduction factor ϕ (LSD)
(i) Bolted connections	
Tear-out	0.60 or 0.70
Net section tension	0.80
Bearing	0.60
Bolts in shear	0.80
Bolts in tension	0.80
(ii) Rupture	
Shear rupture	0.75
Tension rupture	0.90
Block shear rupture	0.80

2.2.2. The American Iron and Steel Institute AISI S100-16 Supplement 3 (2022)

2.2.2.1 General requirements

The design provisions in AISI S100:16 (R2020) w/S3-22 [1] apply to steel-to-steel bolted connections used for CFS structural members in which the thickness of the thinnest connected part is 4.76mm or less. Furthermore, in this design provision, the minimum distance between centres of bolt holes and the distance from the centre of any standard hole to the end or other boundary of the connecting member are similar to that in the AS/NZS 4600:2018 [2]. The minimum size of bolt holes is also summarised in Table 2.5.

Table 2.5: Minimum size of bolt holes (AISI S100:16 (R2020) w/S3-22 [1])

Nominal bolt diameter d_f (mm)	Standard hole diameter d_h (mm)	Oversized hole diameter d_h (mm)	Short-slotted hole dimensions (mm)	Long-slotted hole dimensions (mm)
$d_f < 12$	$d_f + 1.0$	$d_f + 2.0$	$(d_f + 1.0)$ by $(d_f + 6.0)$	$(d_f + 1.0)$ by $2.5d_f$
$12 \leq d_f \leq 20$	$d_f + 2.0$	$d_f + 4.0$	$(d_f + 2.0)$ by $(d_f + 6.0)$	$(d_f + 2.0)$ by $2.5d_f$
$20 < d_f < 24$	$d_f + 2.0$	$d_f + 6.0$	$(d_f + 2.0)$ by $(d_f + 8.0)$	$(d_f + 2.0)$ by $2.5d_f$
$d_f = 24$	27	30	27 by 32	27x60
$d_f > 24$	$d_f + 3.0$	$d_f + 8.0$	$(d_f + 3.0)$ by $(d_f + 10.0)$	$(d_f + 3.0)$ by $2.5d_f$

2.2.2.2 Design shear capacity for bolted connections in shear

Bearing capacity of the bolted connections in shear with and without considering bolt hole deformation in AISI S100:16 (R2020) w/S3-22 [1] Specifications is determined similar to AS/NZS 4600:2018 [2] while the bearing factor (C) is further extended in the cases of connections with oversized or short-slotted holes, as in Table 2.6.

Table 2.6: Bearing factor (C) (AISI S100:16 (R2020) w/S3-22 [1])

Thickness of connected part t (mm)	Connections with standard holes		Connections with oversized or short-slotted holes	
	Ratio of fastener diameter to member thickness (d_f/t)	C	Ratio of fastener diameter to member thickness (d_f/t)	C
$0.61 \leq t < 4.76$	$d_f/t < 10$	3.0	$d_f/t < 7$	3.0
	$10 \leq d_f/t \leq 22$	$4 - 0.1(d_f/t)$	$7 \leq d_f/t \leq 18$	$1 + 14/(d_f/t)$
	$d_f/t > 22$	1.8	$d_f/t > 18$	1.8

The nominal bolt strength (P_n) resulting from shear or tension is determined as follows

$$P_n = A_b f_n \quad (2.10)$$

in which A_b is the gross cross-sectional area of the bolt; f_n denotes the nominal strength of a bolt. $f_n = f_{nv}$ when bolts are subjected to shear only while $f_n = f_{nt}$ when bolts are subjected to tension only; f_{nv} is the nominal shear strength of a bolt; and f_{nt} is the nominal tensile strength of a bolt.

2.2.2.3 Rupture in bolted connections

In the current cold-formed steel design specification from the USA (AISI S100:16 (R2020) w/S3-22 - Section J6.1 [1]), the nominal strength of bolted connections under pure shear rupture (R_{nv}) is calculated as follows

$$R_{nv} = 0.6 f_u A_{nv} \quad (2.11)$$

where f_u is the tensile strength of the connected parts; and A_{nv} is the net area subject to shear (parallel to force). For a connection where each individual fastener pulls through the material towards the limiting edge

$$A_{nv} = 2nt e_{net} \quad (2.12)$$

in which n denotes the number of fasteners on the critical cross-section; t is the base steel thickness of the section; and e_{net} is the clear distance between the end of the material and the edge of the fastener hole or weld. In the case of coped beam-end connection

$$A_{nv} = (h_{wc} - n_b d_h) t \quad (2.13)$$

where h_{wc} is the coped flat web depth; n_b denotes the number of fasteners along the failure path; d_h is the diameter of the hole; and t is the thickness of the coped web.

Under pure tension rupture, the nominal strength of bolted connections (R_{nt}) shall be calculated from AISI S100:16 (R2020) w/S3-22 - Section J6.2 [1] as follows

$$R_{nt} = U_{sl} f_u A_{nt} \quad (2.14)$$

in which U_{sl} is the shear lag factor determined in AISI S100:16 (R2020) w/S3-22 - Section J6.2 [1]; and A_{nt} is the net area subject to tension (perpendicular to force).

In addition, the block shear strength of conventional rectangular block shear bolted connections is specified. In particular, the nominal block shear rupture strength (R_n) in AISI S100:16 (R2020) w/S3-22 - Section J6.3 [1] is determined as follows

$$R_n = 0.6f_y A_{av} + U_{bs} U_{sl} f_u A_{nt} \quad (2.15)$$

where $A_{av} = (A_{gv} + A_{nv})/2$ is the active shear area lying in the middle of the net and gross shear areas; f_y is the yield stress of the connected parts; U_{bs} is a non-uniform block shear factor (0.5 for coped beam shear conditions with more than one vertical row of connectors, 1.0 for all other cases); and gross area subject to shear is denoted by A_{gv} .

Further, according to the concepts in the Commentary on AISI S100:16-C (R2020) w/S3-22 - Section J6 [7], a method for determining the block shear strength with staggered holes is proposed based on various cases of potential failure paths as shown in Figure 2.3. For the staggered paths (i.e., the combination of tension and shear behaviour), Cochrane's formula given in Equation (2.16) is utilised as in AISI S100:16 (R2020) w/S3-22 - Section J6.2 [1] to treat them as tension paths by adding both stagger and gauge terms. The resulting Equation (2.17) specifies $U_{sl} = (0.9 + 0.1d_f/s_f)$ for sheet steel connections not having staggered hole patterns and $U_{sl} = 1.0$ for staggered holes. Consequently, the proposed equations do not provide a consistent methodology for the staggered failure paths which have a shear component as well as tension.

$$A_{nt} = n \left(g - d_h + \frac{(s')^2}{4g + 2d_h} \right) t \quad (2.16)$$

$$P_{nt} = f_u U_{sl} A_{nt} \quad (2.17)$$

where P_{nt} is the nominal tensile rupture strength; t is the thickness of steel sheet; d_h is the diameter of the hole; U_{sl} is the shear lag factor; g is the gauge and s' is the stagger as shown in Figure 2.3. n is the number of gauges g .

Both AS/NZS 4600:2018 [2] and the AISI S100:16 (R2020) w/S3-22 [1] achieve better results for determining the nominal block shear rupture strength in comparison with the AISI S100:16 [17] using the active shear area (A_{av}) instead of the gross area (A_{gv}) or the net area (A_{nv}). However, neither of these design equations (i.e., Equation (2.8) and Equation (2.15)) do not consider the effect of the curling on the shear strength and behaviour of the bolted connections.

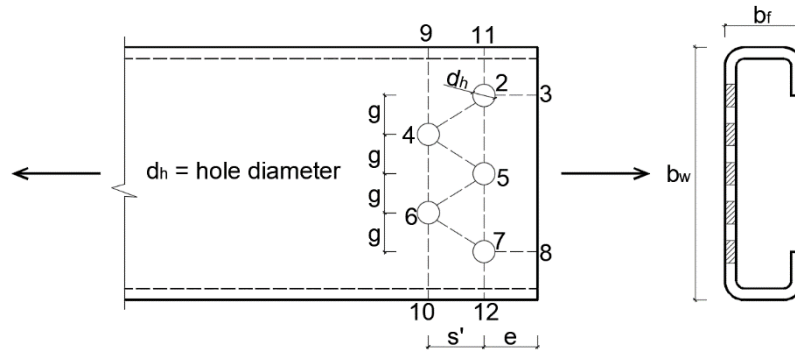


Figure 2.3: Potential failure paths of stiffened channel (According to AISI S100:16-C (R2020) w/S3-22 - Section J6 [7])

2.2.2.4 Resistance factor and safety factor

The AISI S100:16 (R2020) w/S3-22 [1] Specification uses both the LRFD and the historic Allowable Stress Design (ASD) which adopts a safety factor (Ω). The resistance factor (ϕ) is calculated similar to AS/NZS 4600:2018 [2] whereas the safety factor (Ω) is determined on the basis of Equation (2.18). The values of resistance factors and safety factors are listed in Table 2.7.

$$R_d = \frac{R_u}{\Omega} \quad (2.18)$$

Table 2.7: Resistance factor (ϕ) and safety factor (Ω) of bolted connections (AISI S100:16 (R2020) w/S3-22 [1])

Design capacity	Safety factor Ω (ASD)	Resistance factor ϕ (LRFD)
(i) Bolted connections		
Bearing without considering bolt hole deformation	2.50	0.60
Bearing with considering bolt hole deformation	2.22	0.65
Bolts in shear	2.00	0.75
Bolts in tension	2.00	0.75
(ii) Rupture		
Shear rupture	2.00	0.75
Tension rupture	2.00	0.75
Block shear rupture	2.00	0.75

2.2.3. The Eurocode EN 1993-1-8:2005

Provisions for the design of bolted connections in Eurocode 3 - Design of steel structures - Part 1-8: Design of joints [15] illustrates that block tearing involves failure in shear at the row of bolts along the shear face of the hole group accompanied by tensile rupture along the line of bolt holes on the tension face of the bolt group, as demonstrated in Figure 2.4.

According to Eurocode 3 - Design of steel structures - Part 1-8: Design of joints [15], the block tearing resistance (R_n) for a symmetric bolt group subject to concentric loading is given by

$$R_n = \left(1/\sqrt{3}\right) f_y A_{nv} + f_u A_{nt} \quad (2.19)$$

Nevertheless, for a bolt group subject to eccentric loading, the block shear tearing resistance (R_n) is established by

$$R_n = \left(1/\sqrt{3}\right) f_y A_{nv} + 0.5 f_u A_{nt} \quad (2.20)$$

The design Equations (2.19)-(2.20) also do not include explicit equations for staggered hole block shear failure and bolted connections under the curling effect as discussed above.

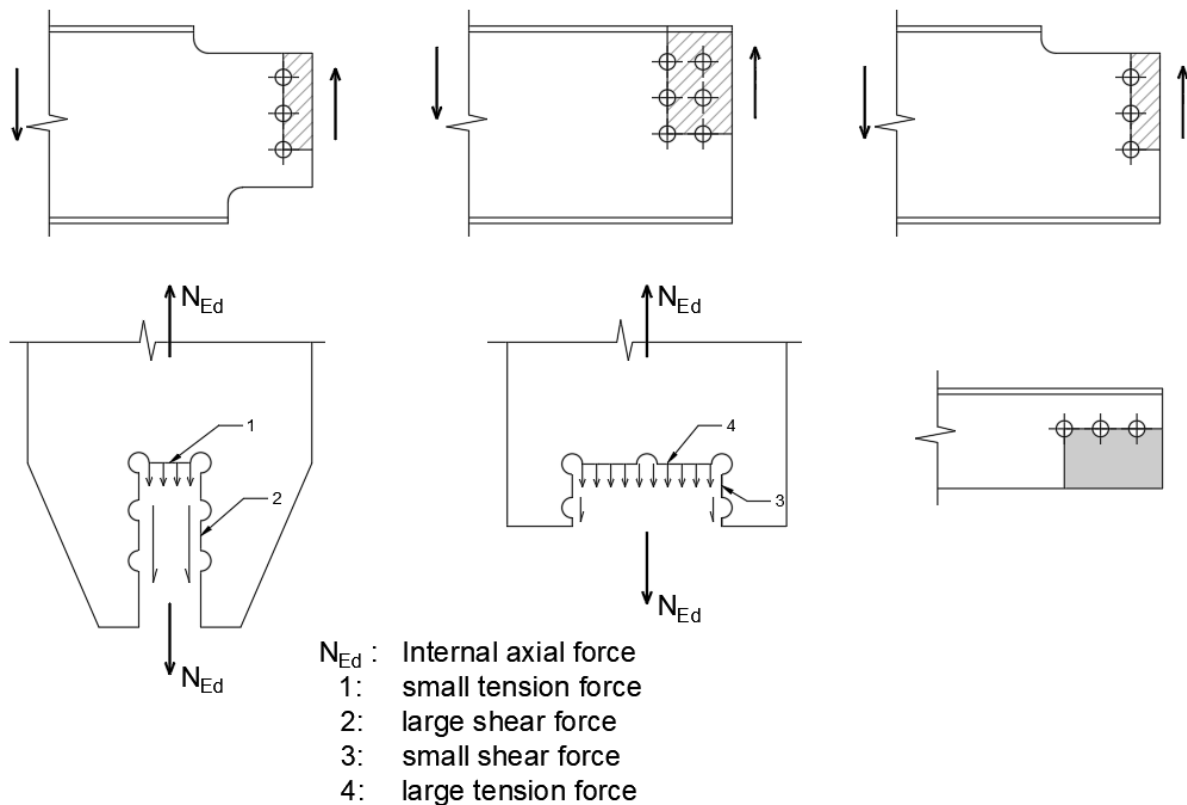


Figure 2.4: Block shear tearing (According to Eurocode 3 - Design of steel structures - Part 1-8: Design of joints [15])

2.3. STUDIES INTO BEHAVIOUR OF STRUCTURAL BOLTED CONNECTIONS IN SHEAR

The structural behaviour of bolted connections in CFS members is different from that in HRS members due to the characteristic properties and higher slenderness. Prior to 1980, the design of bolted connections was originally developed on the basis of the test results by Winter [18], [19]. The results of 547 tests of light-gage steel connections with ordinary bolts (Winter [18]) demonstrated four distinct kinds of connection failures as designated by I, II, III, IV and combined actions of I-III in Figure 2.5. The Failure Type I was typical for the tear-out fracture in the bolted connections with relatively short end distances (the distance measured in the line parallel to the force from the centre of an outer bolt hole and the end of the connected part) while the bearing failure occurred in the Failure Type II when the end distances were relatively long. In addition, the connections failed by the net section tension failure mode (Failure Type III) in the case of connections using the relatively short edge distance (the distance measured in the line perpendicular to the force from the centre of an outer bolt hole and the end of the connected part). The shear fracture in bolt occurred in the Failure Type IV. Consequently, they provided four simple equations for determining failure loads corresponding to each type of above failure modes. Winter [19] further extended 476 additional tests made on similar connections, but with high strength, high torqued bolts in order to investigate in what manner such connections would perform differently from those made with ordinary bolts, and what possible advantage could be gained from using these special bolts in light-gage, thin-walled steel constructions.



Figure 2.5: Types of failures (Winter [18])

Additional research was performed in the United States by Yu [16] not only to further verify the design equations proposed by Winter [18], [19] but also to summarise the changes of the design criteria and to document the research data used for the revision of the AISI Specification [20]. The design equation for bearing of bolted connections with washers was investigated by Rogers and Hancock [21] for G300 and G550 steels and further extended to bolted connections without washers and to the inside sheet of double shear connections with or without washers. Further, the research at the University of Missouri-Rolla by LaBoube and Yu [22] demonstrated the presence of hole elongation prior to reaching the limited bearing strength of a bolted connection. They adopted an elongation of 6.4mm as the acceptable deformation limit then the design equations for bearing strength with consideration of bolt hole deformation have been established in AISI S100:16 (R2020) w/S3-22 [1] (Equation (2.4)).

Thanks to significant capability of the FEM procedure, Salih et al. [23] investigated numerically the net section failure in stainless steel bolted connections. Numerical models were compared with available test data and appropriate failure criteria investigated to serve for validation. Furthermore, their study showed that stainless steel has enough ductility to accommodate extensive redistribution of stresses prior to fracture based on a change of various configurations in the connections.

2.4. STUDIES INTO RUPTURE STRENGTH OF BOLTED CONNECTIONS

2.4.1. Rupture in conventional bolted connections

Connections may be subject to shear rupture, tension rupture and block shear rupture or any combinations of these failures in shear depending on the relationship of the connectors to the connection geometry and loading direction. In particular, block shear is a rupture or tearing phenomenon of a group of bolts where the block shear capacity includes two components (i.e., the shear strength on the failure path parallel to the force and the tension strength on the failure path perpendicular to the force). In the literature, Birkemoe and Gilmor [24] and Hardash and Bjorhovde [25] conducted a study on the block shear strength of bolted connections. In their research, various possibly fracture paths of the group of bolts were investigated corresponding to different equations for determining the shear strength of the connections. Nevertheless, these equations were validated against the test results of the hot-rolled steel bolted connections. In addition, Kulak and Grondin [26] provided a research paper including a report of test database for block shear rupture strength in hot-rolled steel bolted connections.

Later, Teh and Clements [27] further studied the phenomena and mechanisms for block shear failures of bolted connections in cold-reduced steel sheets. Their study indicated that a

tensile rupture and shear yielding mode is the only feasible mechanism for limit state design of traditional block shear failure. In addition, an in-plane shear lag factor in the tension zone (further studied by Teh and Yazici [28]) and active shear resistance planes were investigated based on the tensile rupture and shear yielding mechanism resulting in a more accurate and consistent equation for predicting the block shear capacity of the connections. Consequently, block shear limit state was proposed by Teh and Clements [27] which has been included in AS/NZS 4600:2018 [2] and AISI S100:16 (R2020) w/S3-22 [1]. Similar behaviour was also observed by Clements and Teh [29] using geometrically and materially nonlinear contact finite brick element analysis to confirm the location of the active shear planes.

More recently, Mai et al. [30] further investigated block shear strength and behaviour of cold-reduced G450 sheet steel bolted connections utilising the Digital Image Correlation (DIC) [12]. In conjunction with the conventional experimental method, the DIC provides detailed failure information on connection behaviour and its correlation with macroscopic material tests. In addition, the experimental results illustrated an overview of the failure modes of bolted connections. Consequently, the features of the new block shear formulas recently employed in the AISI S100:2016 [17] and AS/NZS 4600:2018 [2] were clarified. Their results demonstrated that the strain in the compression zone increased monotonically and finally converged to a peak load resulting in no fracture in this zone while that in the tension zone had a different feature. At the peak load, the strain experienced a softening phase. Moreover, a non-uniform strain distribution in this zone occurred because of the larger distance between the two bolts. In the same year, Ahmed and Teh [31] studied the influence of bolt threads on the stiffness of bolted shear connections through experiments and finite element procedures. They also concluded that the reduction of the initial stiffness of the threaded specimens occurred as a result of the threads cutting into the plates leading to an increment of the initial displacement. However, the same factor can decrease the crimping of the plate material downstream of the bolt in the case of a bolted hollow section, improving the final stiffness of the threaded bolted connections as they approached their ultimate loads. Furthermore, their study illustrated the minor influence of end and edge distances between bolts on the elastic stiffness of the connections. Consequently, ad hoc stiffness formulae were employed for shank and thread bolted connections.

The effect of material ductility and temperature on the block shear capacity of bolted connections was investigated by Cho et al. [32]. They concluded that the code equations for the block shear failure utilise various assumptions regarding the extent of strain hardening. In particular, at room temperature, shear strain hardening of the ultra-high strength steel specimens was very small because of the low ductility while full shear strain hardening was employed for

mild steel and aluminium bolted connections. However, at high temperature above 300°C , shear strain hardening was significant for the ultra-high strength steel specimens.

The behaviour of cold-formed steel bolted connections under shear was also studied by Chung and Ip [33] using a FEM procedure. In addition, the influence of configurations was investigated based on a parametric study resulting in a guideline for empirical design. By means of comparison between the FEM and codified design rules, they illustrated that the design rules are not applicable in the case of high strength steels for bolted connections due mainly to a reduction of ductility. Finally, a calibrated FEM model was implemented to propose a semi-empirical design equation for bearing resistance of bolted connections to overcome that problem.

2.4.2. Rupture in staggered hole bolted connections

The presence of staggered or diagonal patterns in a bolted connection has long been recognized as increasing the net section area for the limit state of rupture. It was first analytically researched by Cochrane [34]. In his research, the rupture capacity in the staggered or diagonal patterns was treated as tension capacity through the stagger and gauge terms (Equation (2.16)).

Recently, the tension capacity of staggered bolted connections in cold-reduced steel sheets was studied by Teh and Clements [35]. Their test results pointed out that the simplification of Cochrane's original formula (Equation (2.21)) into the standard formula leads to overestimations of the net section area (A_{nt}), while there is little to be gained from the trivial simplification.

$$A_{nt} = w - n_s d_h + \sum \frac{(s')^2}{4g} \quad (2.21)$$

where w is the width of the plate; n_s is the number of bolts in the considered section.

In addition, they demonstrated that the assumption implicit in Cochrane's formula (i.e., stresses are uniformly distributed over the net section) is over-optimistic for connections in steel in the case of low ductility causing unconservative estimates for the tension capacity if the in-plane shear lag is excluded. By means of laboratory test results, they proposed an accurate and consistent Equation (2.22) that included the in-plane shear lag for analyzing the net section tension capacity (P_{nt}).

$$P_{nt} = \left(0.9 + 0.1 \frac{d_h}{w} \right) f_u t \left[w - \max \left((n_s - 1) d_h, n_s d_h - \sum \frac{(s')^2}{4g + 2d_h} \right) \right] \quad (2.22)$$

A block shear design equation was examined by Teh and Yazici [36] based on an experimental program of unconventional bolted connection. The laboratory tests included

specimens failing in the staggered block shear and split block shear. In particular, the dominant mechanism for block shear failures of staggered hole bolted connections is shear yielding and tensile rupture mechanism instead of shear rupture and tensile yielding mechanism. The proposed equation was found to be accurate for determining the block shear strength of staggered bolted connections. A similar study was also investigated by Bharath et al. [37].

2.4.3. Rupture in bolted connections under curling effect

Some features of the thin-walled structural members also change the behaviour of the bolted connections. When the bolted connections are used to attach thin and thick plates and the end distance between the bolt holes and the free edge of the thin plate is quite long, a curling phenomenon may occur in the bolted connections, affecting the shear strength of the connections. In the literature, the effect of the curling on the block shear and tear-out behaviour of the cold-formed steel bolted connections has not been properly investigated so that structural engineers may need to implement independently experiments to establish design models. The effect of curling in carbon steel bolted connections was studied by Kim et al. [3]. In their work, single shear bolted connections fabricated with four bolts and thin-walled carbon steel were tested to investigate the fracture mechanism and the curling influence on the ultimate strength. Their research implied that the occurrence of curling could decrease the ultimate strength of the single shear bolted connections corresponding to both the plate thickness and the end distance. An influence coefficient for the curling phenomenon was added into the original block shear capacity to take into account of the curling effect.

Further, the curling effect of mechanically fastened joints in cold-formed austenitic stainless steel was investigated using FEM by Kim and Kuwamura [4]. Their study focused on the ultimate behaviour and the effect of curling on the ultimate strength in bolted joints fabricated with one bolt based on various end distances, edge distances and plate thicknesses. Their study implied that the design manual in SSBA 2006 (Stainless Steel Building Association) [38] provided a reasonable prediction for ultimate strength of bolted joints without curling in comparison with ultimate strength predicted by AISI S100:2016 [17]. Moreover, for fastened joints with severe local buckling or out-of-plane deformation (hereafter, curling), the SSBA manual also overestimated the ultimate strength similar to the current design standards. In addition, the condition of curling occurrence differs from the above stated variables. Curling reduced the ultimate strength by 19–39% compared to bolted joints without curling. Consequently, a coefficient accounting for the curling effect was added into the original block shear capacity.

2.5. DEVELOPMENT OF DUAL ACTUATOR SETUP FOR TESTING SHEAR BEAM-END CONNECTIONS

The earliest research on the beam-to-column connections for HRS members was conducted by Astaneh-Asl et al. [39]. They believed that most failure modes of simple shear connections are related to the combination of shear and rotation effects. The test apparatus is demonstrated in Figure 2.6 where Actuator S which was close to the connection was used to apply the vertical shear, while Actuator R provided beam end rotation. Two actuators were controlled independently which allowed any prescribed shear-rotation combinations to be applied to the connections so that the dual test-rig provided the most realistic shear-rotation relationship in the inelastic regime in comparison with various test rigs used by other researchers.

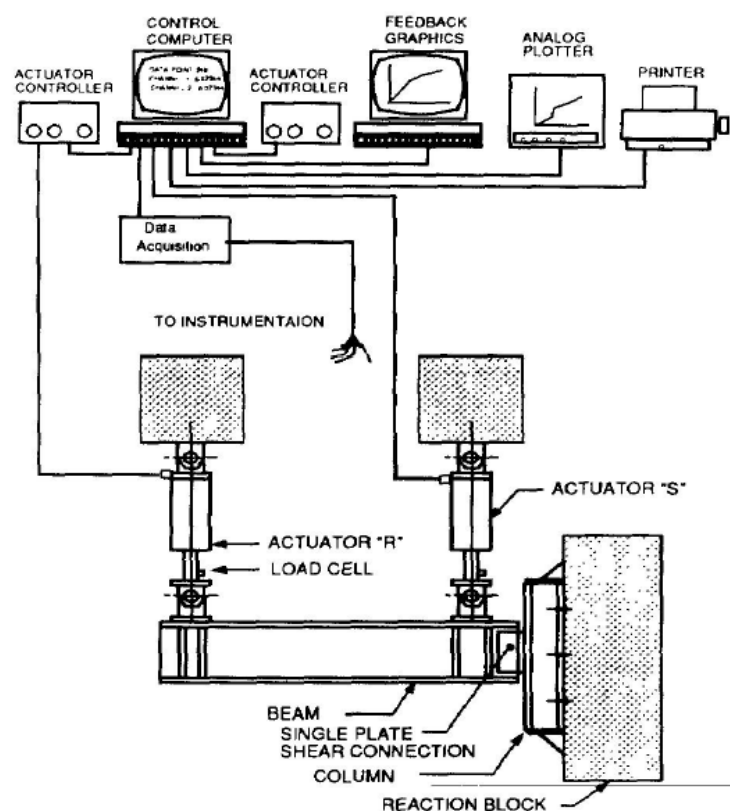


Figure 2.6: Dual actuator beam-to-column connection test (Astaneh-Asl et al. [39])

In conventional design, shear beam-end bolted connections are assumed to be simple connections which only transfer shear and have a negligible resistance to rotation. However, Astaneh-Asl et al. [6] demonstrated that simple connections in floor systems are commonly subjected to a combined effect of shear and rotation caused by gravity loads and the deflection of the beam or building sway. In their study, a plate was welded to a support at one edge and bolted to a beam web. By means of experimental and analytical studies, a new design procedure was established in their research to analyze limit states in design of bolts, plates, welds, beams without block shear failure in the beams. In addition, to investigate the shear-rotation relationship, they

applied shear forces and corresponding rotations to the connections. They revealed that some simple connections could actually develop beam-end moments up to 20% of the moment of a fixed-ended beam.

Later, Gong [40], [41] and [42] further extended the applications of the dual-actuator test setup to investigate the behaviour of bolted shear connections using angle cleats and shear tabs. Their test apparatus is demonstrated in Figure 2.7 where the shear force was mainly created by Actuator Jack 1 which was close to the connection while Actuator Jack 2 was used to provide rotation of the beam. Actuator Jack 3 was used to support the beam-end.

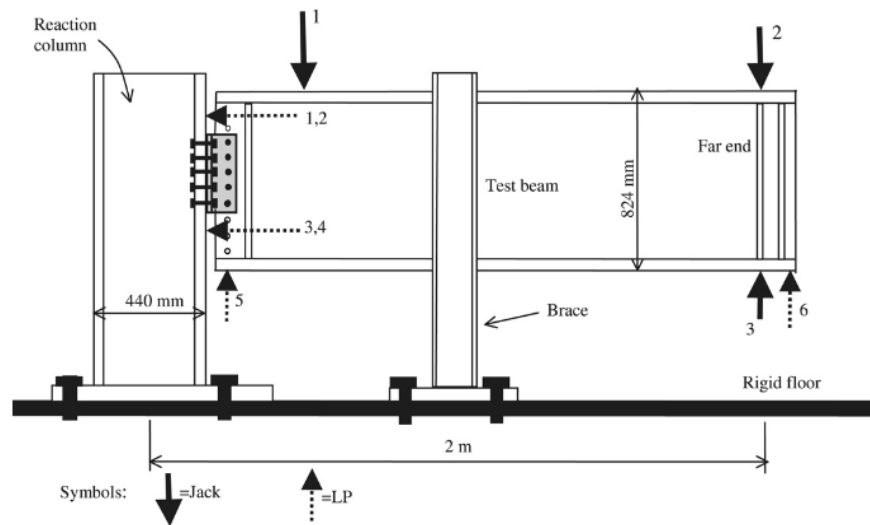


Figure 2.7: Elevation of test setup using dual actuator (Gong [40])

Similar test configurations for hot-rolled steel shear bolted connections were investigated by Rogers et al. [43] and Thomas et al. [44]. The test apparatus in Rogers et al. [43] study (Figure 2.8) was similar to those established in the previous studies for a dual actuator test-rig. A combination of axial load, shear load, and rotation was used based on three actuators (Figure 2.9) in the Thomas et al. [44] study to investigate the behaviour of full-scale shear connections. In their experimental program, three actuators were controlled independently where Actuator 2 was used to create a rotation of 0.03 rad whereas the horizontal load was applied by Actuator 3. Actuator 1 adjacent to the connections was used to mainly create the shear force. They believed that their test setups and loading protocols could reasonably represent the severe loading conditions in practice which provided valuable insights into the behaviour and strength of single-plate connections in eight different limit states.

Recently, Wen and Mahmoud [45] performed a numerical study to simulate block shear fracture in gusset plate and coped beam connections. They also pointed out that the shear fracture planes occurred between the net and gross sections similar to the work of Teh and Clements [27].

Furthermore, in recent studies for shear beam-end connections joining CFS channels to a rectangular hollow section or a tubular column, Huynh et al. [46] and Truong et al. [47] adopted similar setups (i.e., Astaneh-Asl et al. [6], Rogers et al. [43] and Thomas et al. [44]) for screwed connections (Figure 2.10) and Power-Actuated Fastener (PAF) connections (Figure 2.11), respectively. By comparison, CFS members do not have a plastic collapse but may have excessive deflections or fail in a mode such as distortional buckling which is totally different from HRS connections.

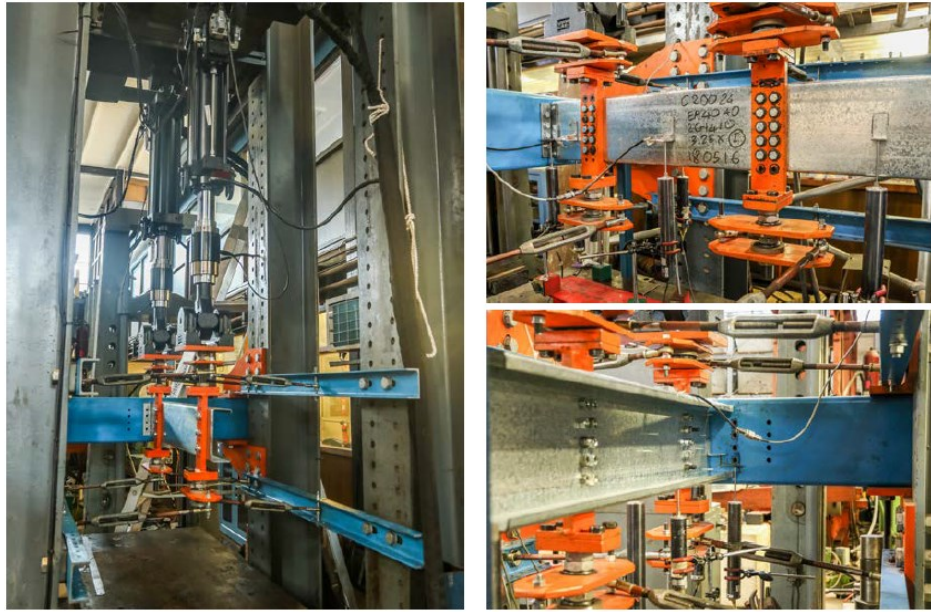


Figure 2.10: Hybrid test setup for screwed connection from cold-formed channel to RHS beam (Huynh et al. [46])

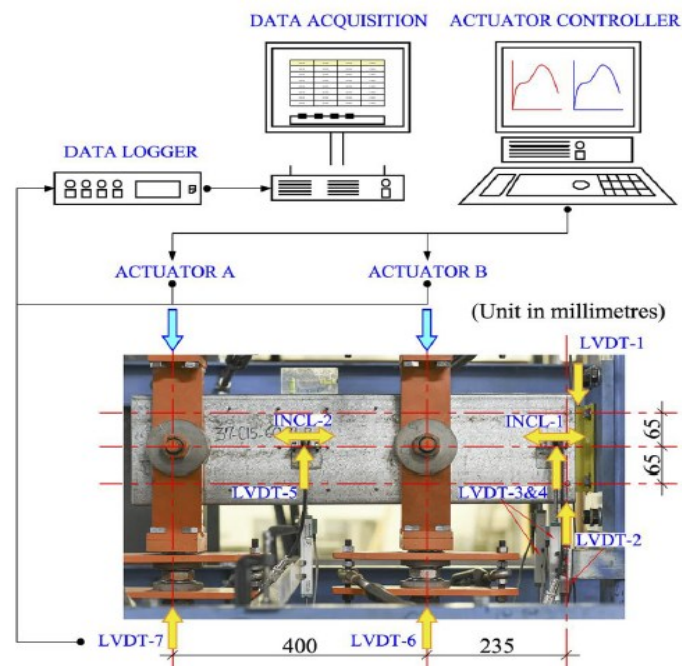


Figure 2.11: Hybrid test setup for PAF connection from cold-formed channel to tubular column (Truong et al. [47])

2.6. SUMMARY

This chapter has provided a review of the important literature and valuable insights into the behaviour and rupture strength of bolted connections mainly failing in block shear. Although design provisions for bolted connections have been included in many standards and specifications, the applications of different types of connections such as beam-to-beam or beam-to-column connections was not fully explored for CFS bolted connections especially in high strength steel and requires further investigation. Furthermore, in the latest cold-formed steel design standards and specifications from AISI S100:16 (R2020) w/S3-22 [1] and AS/NZS 4600:2018 [2], the design equations are only specified for simple bolted connections in shear and there is a lack of corresponding test results for CFS shear beam-end bolted connections. In addition, in some cases of special configurations with staggered holes or with relatively long end distance (i.e., the distance between outer bolt holes and free edge), structural tests are required without appropriate guidelines from the current design standards, specifications and literature. Therefore, it is necessary to carry out more experiments to investigate the rupture strength and fracture behaviour of CFS bolted connections especially in thin plates and high strength steel to provide guidelines for structural engineers to employ into practical applications.

CHAPTER 3

EXPERIMENTAL INVESTIGATION OF STAGGERED HOLE BOLTED CONNECTIONS

3.1. INTRODUCTION

This chapter demonstrates a complex behaviour of high strength steel staggered hole bolted connections failing in block shear by carrying out an experimental program comprised of 24 tests using G450 sheets. Digital Image Correlation (DIC) [12] was employed to capture both the full field of the displacement and the block shear failure mode of the staggered hole bolted connections during testing. Consequently, a design equation for block shear strength with staggered holes is proposed accounting for a combination of shear rupture and tension rupture in the staggered zones. It demonstrates a better correlation with the experimental results than current design standards such as AISI S100:16 (R2020) w/S3-22 [1] and AS/NZS 4600:2018 [2]. A reliability analysis is included in this chapter.

3.2. EXPERIMENTAL INVESTIGATION OF STAGGERED HOLE BOLTED CONNECTIONS

The experimental program was performed in the JW Roderick Structures Laboratory in the School of Civil Engineering, the University of Sydney using an MTS Sintech 65/G testing machine. In addition, the AISI S905-13: Test Standard for Cold-formed Steel Connections [8], Specification for Structural Joints Using ASTM A325 or A490 Bolts [9], Research report RP01-4 – AISI: Testing of Bolted Cold-Formed Steel Connections in Bearing (With and Without Washers) [10], and the European Recommendations – ECCS TC7 TWG 7.10: The Testing of Connections with Mechanical Fasteners in Steel Sheeting and Sections [11] were employed as the testing methodologies.

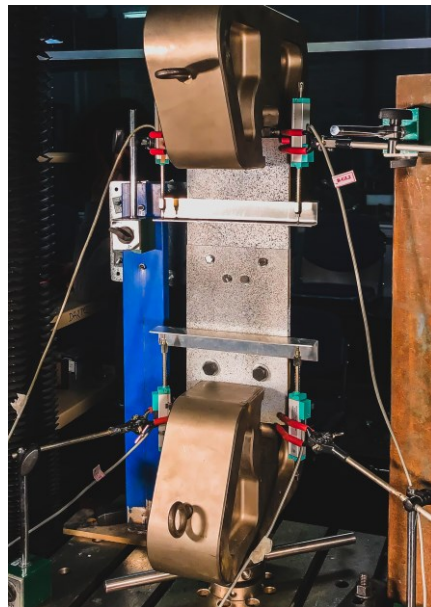
All specimens were fabricated with thickness of 1.9mm and 2.4mm using cold-formed G450 steel sheets. Average material properties of the steel sheets were obtained from coupon tests taking in the longitudinal and transverse directions, as can be seen in Table 3.1. Details of the coupon tests including yield stress, tensile strength and elongation are given in Appendix B. In addition, actual yield stress and tensile strength were used to properly evaluate the actual shear strength of the bolted connections which is then properly used for the verification procedure of the proposed design models. High strength steel bolts (i.e., Grade 8.8) with two diameters (i.e., 8mm and 12mm) were used in the tests in order to mitigate the shear failure in the bolts, and to investigate the influence of hole sizes on the staggered hole bolted connections. In particular, the

diameter of bolt holes is 2mm larger than that of their corresponding nominal bolt diameters. For the 12mm bolts, the 14mm hole is a standard hole according to AISI S100:16 (R2020) w/S3-22 [1] and for the 8mm bolts, the 10mm hole is non-standard. In addition, a non-threaded section on the shank of the bolts was utilised at the interactional areas between the bolts and the steel plates in order to eliminate the curling of the steel plates. Under the DIC technique testing conditions, all specimens were tested without using nuts and washers to capture the whole sample surface during the testing process. In addition, the presence of nuts and washers did not influence the observed behaviour of the connections in shear.

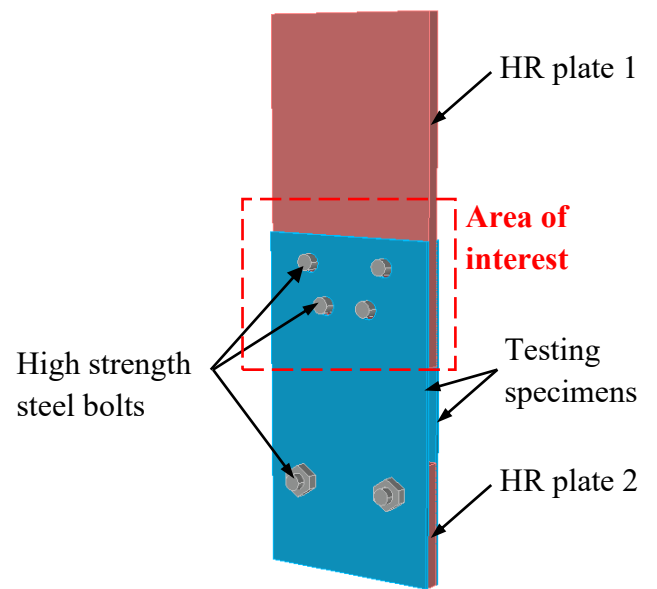
Table 3.1: Average material properties of steel sheets (full test data in Appendix B)

Steel	Nominal thickness (mm)	Base metal measured thickness (mm)	Yield stress f_y Longitudinal (MPa)	Ultimate strength f_u Longitudinal (MPa)	Yield stress f_y Transverse (MPa)	Ultimate strength f_u Transverse (MPa)
G450	1.90	1.89	526.58	551.14	581.06	605.33
G450	2.40	2.37	521.01	547.22	589.45	606.53

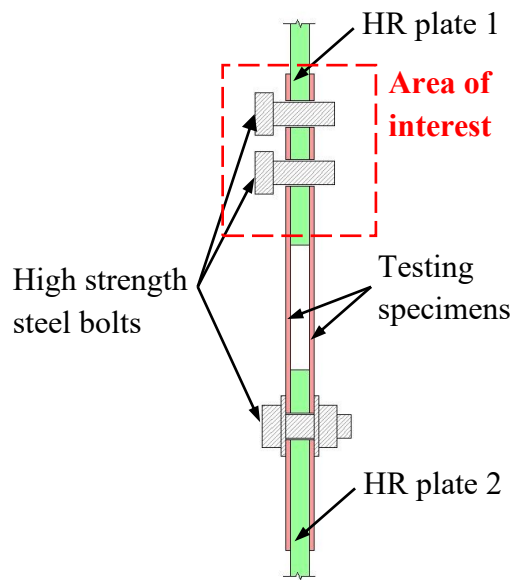
A typical test arrangement of the staggered hole bolted connections is demonstrated in Figure 3.1. In particular, two outside cold-formed steel sheets were connected to two inside 10mm hot-rolled steel plates (i.e., HR Plate 1 and 2) by means of the high strength steel bolts. Furthermore, the upper grip and the lower grip of the MTS machine played a necessary role to hold the upper hot-rolled steel plate and the lower hot-rolled steel plate, respectively. A rate of 0.2mm/min of the crosshead speed was used to ensure a quasi-static response in conjunction with the displacement-control technique. The load results from the MTS machine and the displacement from the linear variable displacement transducers (LVDTs) were recorded simultaneously by a data logger. In addition, aluminium equal angles were attached to the sheet as supports for measuring displacement with the LVDTs. At least two tests were used for each type of configuration. The geometric details of the test specimens are given in Table 3.2 and Figure 3.2. Typical test results of staggered hole bolted connection are illustrated in Figure 3.3.



(a)



(b)



(c)

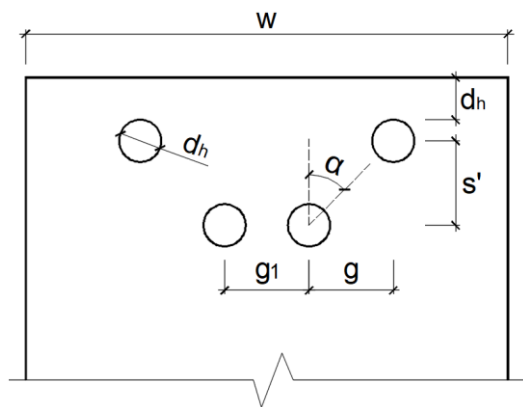


(d)

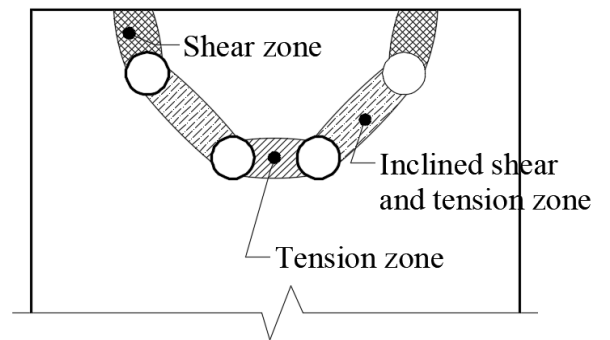
Figure 3.1: Schematic of (a) test set-up, (b) test specimen, (c) cross-section of test specimen and (d) DIC set-up of a staggered hole bolted connection

Table 3.2: Specimen configurations

Specimen	Nominal thickness t (mm)	Nominal width w (mm)	Nominal hole size d_h (mm)	Nominal bolt size d_f (mm)	Gauge g_1 (mm)	Gauge g (mm)	Stagger s' (mm)	Angle α (°)
A2.4-12-00	2.40	160	14	12	28	0	28	0
A2.4-12-27	2.40	160	14	12	28	14	28	27
A2.4-12-45	2.40	160	14	12	28	28	28	45
A2.4-08-00	2.40	160	10	8	18	0	18	0
A2.4-08-29	2.40	160	10	8	18	10	18	29
A2.4-08-45	2.40	160	10	8	18	18	18	45
A1.9-12-00	1.90	160	14	12	28	0	28	0
A1.9-12-27	1.90	160	14	12	28	14	28	27
A1.9-12-45	1.90	160	14	12	28	28	28	45
A1.9-08-00	1.90	160	10	8	18	0	18	0
A1.9-08-29	1.90	160	10	8	18	10	18	29
A1.9-08-45	1.90	160	10	8	18	18	18	45



(a)



(b)

Figure 3.2: Schematic of (a) staggered hole bolted connections configuration and (b) zones of strain extension



(a)



(b)

Figure 3.3: Schematic of a staggered hole bolted connection (a) after failing in the tension zone and (b) after failing further in the inclined shear and tension zone

A high-resolution camera (i.e., Nikon D810 body and a 105 mm lens) was utilised for the DIC procedure. This camera was set up perpendicular to the sample surface in a reasonable position to focus on the area of interest. Images were captured automatically during the test with 10-second intervals before post-processing with the GOM correlation software [48]. Furthermore, two lights were pointed perpendicular to the sample surface in order to eliminate the shadow of the bolts. Both the lights and camera were maintained stable throughout the test to achieve a high accuracy of measurement. The correctness of the DIC technique was verified in comparison with the LVDT results as can be seen in Figure 3.4. In particular, it was found by comparing the DIC curve with the reference solution generated from the LVDTs that the two curves are almost indistinguishable.

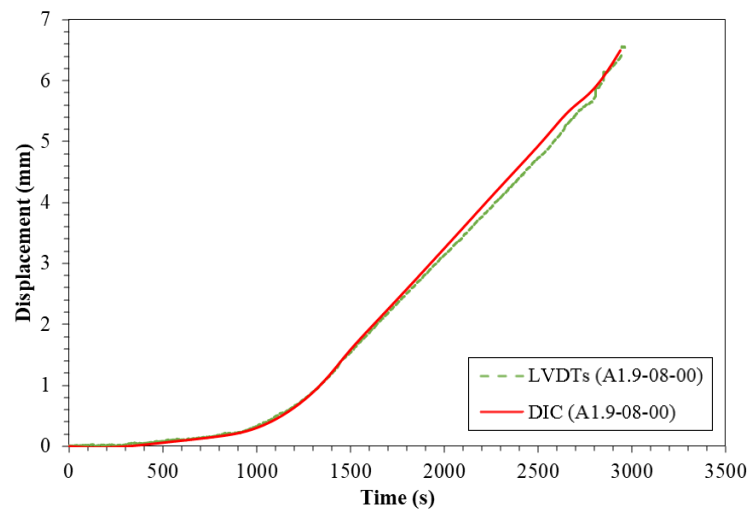


Figure 3.4: Displacement comparison between DIC and LVDTs at the top edge of the specimen A1.9-08-00

3.3. EXPERIMENTAL RESULTS

The test results of the staggered hole bolted connections with various thicknesses (i.e., $t = 1.9, 2.4\text{mm}$) and hole sizes (i.e., $d_h = 10, 14\text{mm}$) are illustrated in this section to investigate both the ultimate block shear strength and the failure modes of the connections. In particular, an average test result between two specimens of each test was used for the thesis and analysis.

3.3.1. Staggered hole bolted connections tests of 2.4mm cold-formed steel sheets

Table 3.3 provides the test results of 2.4mm cold-formed steel staggered hole bolted connections where two tests were conducted for each type of configuration. These dimensions are the same for the repeated tests, in which their test names have a suffix added (i.e., 01, 02, etc.). For example, all the test names A2.4-12-00-01, A2.4-12-00-02 have the same configuration.

It is found in Table 3.3 that all the test results are consistent. Furthermore, the inclined shear significantly affected the ultimate block shear strength of the staggered hole bolted connections

corresponding to a change of the angle α . In addition, the connections achieved the highest block shear strength when the angle $\alpha = 45^\circ$ whereas they only had a small change for the remaining cases (i.e., $\alpha = 0^\circ$; 27° ; 29°). Details of the full set of load-displacement curves are given in Appendix A.

Table 3.3: Results of staggered holes bolted connections of 2.4mm cold-formed sheets

Specimen	Nominal thickness t (mm)	Nominal hole size d_h (mm)	Angle α ($^\circ$)	Experimental results P^T (kN)
A2.4-12-00-01	2.40	14	0	75.19
A2.4-12-00-02	2.40	14	0	73.75
A2.4-12-27-01	2.40	14	27	79.46
A2.4-12-27-02	2.40	14	27	78.59
A2.4-12-45-01	2.40	14	45	105.20
A2.4-12-45-02	2.40	14	45	103.30
A2.4-08-00-01	2.40	10	0	46.85
A2.4-08-00-02	2.40	10	0	46.54
A2.4-08-29-01	2.40	10	29	49.44
A2.4-08-29-02	2.40	10	29	48.95
A2.4-08-45-01	2.40	10	45	64.81
A2.4-08-45-02	2.40	10	45	65.46

Load-displacement behaviour of the three types of configurations (i.e., $\alpha = 0^\circ$; 27° ; 45°) of the staggered hole bolted connections using the M12-high strength steel bolts (e.g., $d_f = 12\text{mm}$) are shown in Figure 3.5a and for the M8-Bolts in Figure 3.6. Clearly, the inclined shear changed the behaviour of these types of connections. Although all three configurations had similar behaviour in the elastic and the plastic ranges, the intermediate range depended on the inclined shear. The relationship between the structural response and the strain distribution for the angle $\alpha = 45^\circ$ is discretized into seven stages in Figure 3.5a & b. In terms of the angle $\alpha = 45^\circ$, the connection in this case was the least ductile after tension fracture although it obtained the highest block shear strength. The block shear strength was up to 25-27% larger than the others. In particular, the strain in the elastic phase (e.g., Stage 1) only localized in front of the four bolt holes before expanding in the hardening range (e.g., Stage 2). In this stage, yielding started to appear in the tension zone between the two lower bolts. This region was more critical for the strain distribution in comparison with the other zones (i.e., the distance between the two lower bolts is smaller than the inclined shear), then when the shear strength reached the peak (e.g., Stage 3), the strain in the tension zone increased significantly while the strain in the inclined shear zones also moved into the hardening range. After failing in the tension zone as a result of critical strain during the period from Stage 4 to Stage 5 the specimen then failed in the inclined shear zone

almost simultaneously (e.g., Stage 6), as illustrated through the steep negative slope in Figure 3.5a. Substantial deformation occurred during shear fracture in the remaining shear zone (e.g., Stage 7).

In the case of the test with angle $\alpha = 0^\circ$ shown in Figure 3.7, after failing in the tension zone between the two lower bolts, the connection experiences a substantial extension of the strain in the shear zone resulting in an improvement of the ductility of the connection. This behaviour of the traditional rectangular block shear strength is also demonstrated in detail by Mai et al [30]. Furthermore, it is noticed that shear failure occurs in the outside zone (i.e, closer from the end) first, followed by the inside shear zone (i.e, farther from the end) fracture, as demonstrated in Figure 3.7a. Meanwhile, the ultimate block shear strength with the angle $\alpha = 27^\circ$ does not have any improvement in comparison with the second case (i.e., the angle $\alpha = 0^\circ$) whereas the behaviour has changed significantly in the fracture stage because of the combination between the tensile and shear behaviour in the inclined shear and tension zone, which also reduces the ductility of the connection. In particular, the connection fails in the tension zone at the same value as the second case before fracturing in the inclined shear zone, and the outside shear zone as shown in Figure 3.8a. It demonstrates an inverse process in the shear zone failure compared with the second case. In addition, the behaviour of this case in the final stage is the same as the first case (i.e., the angle $\alpha = 45^\circ$) while the failure process between the tension zone and the inclined zone was later than for the first one, as demonstrated through the two slopes in Figure 3.5a. It is observed from the failure processes of the three different configurations that the connections were more ductile during shear fracture than tension failure.

The behaviour of these types of configurations were further studied using the M8-high strength steel bolts (e.g., $d_f = 8\text{mm}$) to investigate the influence of hole size on the staggered hole bolted connections. It is to be noted that the hole size in this case is 10mm which is non-standard. It is apparent from Figure 3.6 that the behaviour of the three different configurations had the same trend as that observed in Figure 3.5a. However, the staggered hole bolted connections with the larger hole size (i.e., $d_h = 14\text{mm}$) were more than 20% more ductile than that using smaller hole size (i.e., $d_h = 10\text{mm}$). Likewise, the block shear strength of the smaller hole connections was lower by nearly 39% than that of the larger hole connections, which confirms the enhancement of the shear strength as well as the ductility of the connections by using larger bolts.

Figure 3.8b & Figure 3.9b shown the same behaviour of the failure process as for the staggered hole bolted connections as those depicted in Figure 3.8a & Figure 3.9a, respectively. By comparison, the failure process in the shear zone of the specimen A2.4-12-00 had an inverse trend compared with the specimen A2.4-08-00, as demonstrated respectively in Figure 3.7a & b.

Furthermore, it is demonstrated in Figure 3.7-Figure 3.9 that the fracture lines in both the inclined shear zones and the pure shear zones were significantly correlated to the angle α .

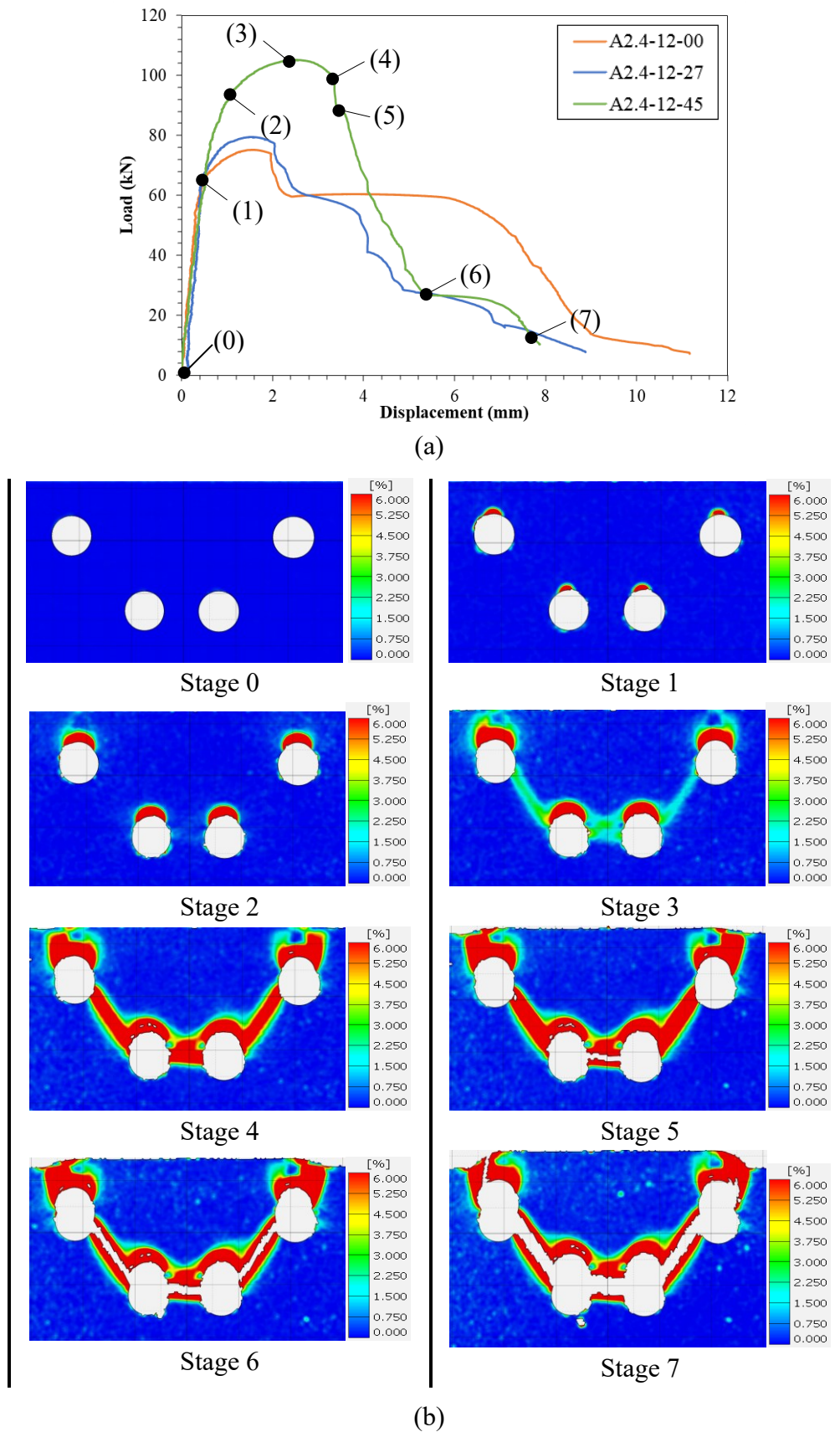


Figure 3.5: (a) Test results of 2.4mm cold-formed steel staggered hole bolted connections using M12-Bolts and (b) Von Mises strain distribution of the specimen A2.4-12-45

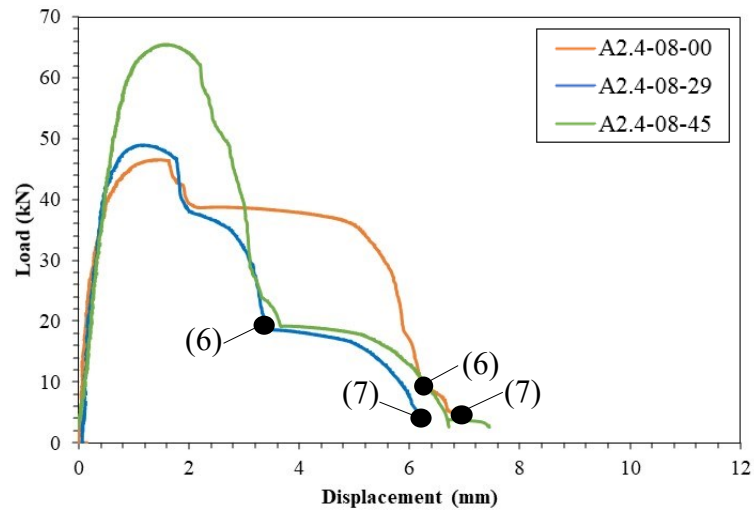


Figure 3.6: Test results of 2.4mm cold-formed steel staggered hole bolted connections using M8-Bolts corresponding to failure process of the specimens A2.4-08-00 and A2.4-08-29

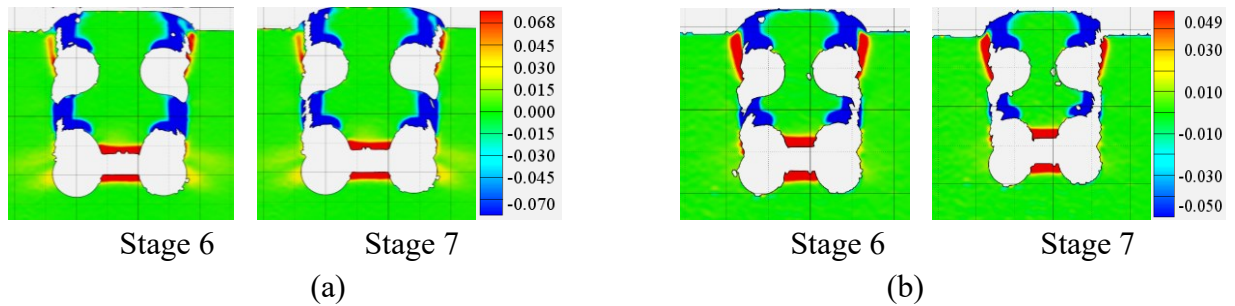


Figure 3.7: True strain distribution of the specimens (a) A2.4-12-00 and (b) A2.4-08-00 using DIC

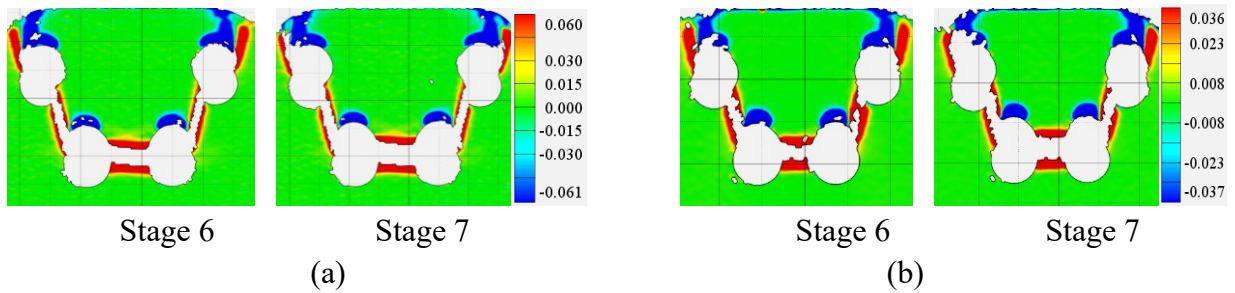


Figure 3.8: True strain distribution of the specimens (a) A2.4-12-27 and (b) A2.4-08-29 using DIC

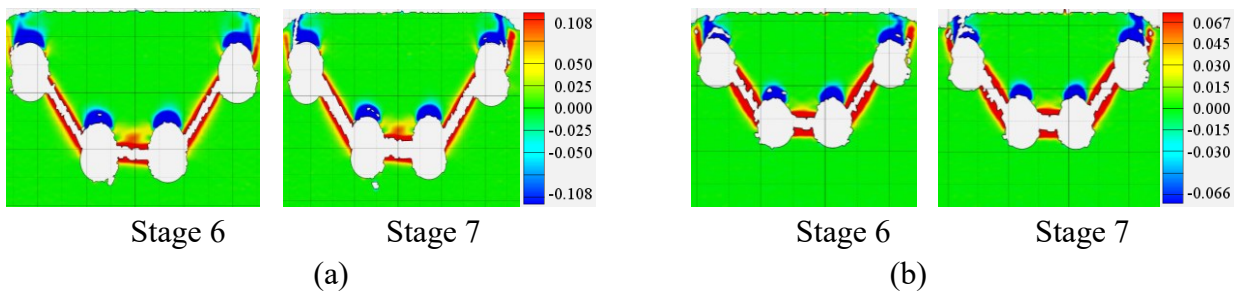


Figure 3.9: True strain distribution of the specimens (a) A2.4-12-45 and (b) A2.4-08-45 using DIC

3.3.2. Staggered hole bolted connections tests of 1.9mm cold-formed steel sheets

By following the same experimental procedure as employed in the previous series of test (i.e., $t = 2.4\text{mm}$), the behaviour of the staggered hole bolted connections using 1.9mm high strength steel sheets was further examined to investigate the influence of thickness on the connections. The test data of the experiments are provided in Table 3.4. It is demonstrated that the hole size and the inclined shear significantly affected the shear strength of the connections in both cases of the M12-high strength steel bolts and the M8-high strength steel bolts. The shear strength of the staggered hole bolted connections increased since the inclined shear and the hole size increased as for the 2.4mm sheets.

Table 3.4: Results of staggered holes bolted connections of 1.9mm cold-formed sheets

Specimen	Nominal thickness t (mm)	Nominal hole size d_h (mm)	Angle α ($^\circ$)	Experimental results P^T (kN)
A1.9-12-00-01	1.90	14	0	58.94
A1.9-12-00-02	1.90	14	0	58.56
A1.9-12-27-01	1.90	14	27	62.12
A1.9-12-27-02	1.90	14	27	63.03
A1.9-12-45-01	1.90	14	45	83.58
A1.9-12-45-02	1.90	14	45	81.51
A1.9-08-00-01	1.90	10	0	36.21
A1.9-08-00-02	1.90	10	0	35.81
A1.9-08-29-01	1.90	10	29	39.05
A1.9-08-29-02	1.90	10	29	38.83
A1.9-08-45-01	1.90	10	45	51.33
A1.9-08-45-02	1.90	10	45	50.75

The behaviour of the staggered hole bolted connections with the three different inclined shear angles using the M12-high strength bolts and the M8-high strength steel bolts are given in Figure 3.10 & Figure 3.11, respectively. Again, the behaviour of the conventional rectangular block shear bolted connections (i.e., the angle $\alpha = 0^\circ$), and the two remaining cases totally followed the first series of tests. Moreover, the ductility of these cases was very similar as for the first series, as demonstrated by comparing Figure 3.5 & Figure 3.10 and Figure 3.6 & Figure 3.11, respectively. It implies that the ductility was independent of the thickness. On the contrary, the shear failure process in the conventional rectangular block shear bolted connections (i.e., Figure 3.12a) had the opposite trend in comparison with the first series (i.e., Figure 3.7a). In particular, the shear fracture occurred in the inside shear zone before propagating to the outside shear zone, as employed in Figure 3.12a. (See Figure 3.13 & Figure 3.14).

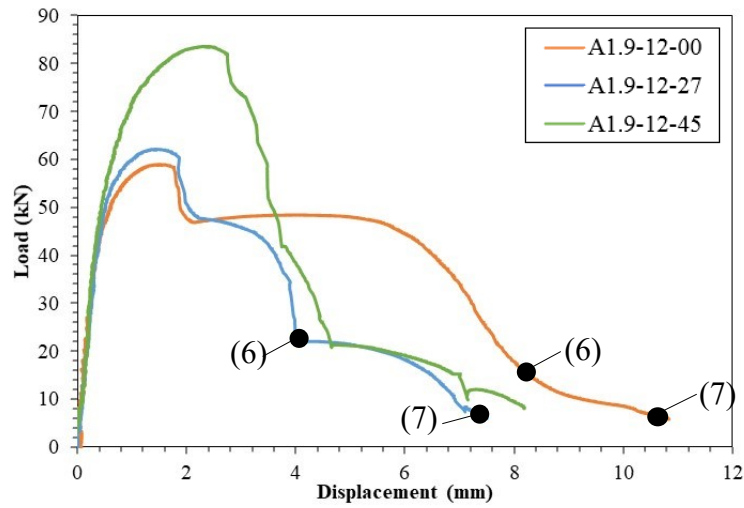


Figure 3.10: Test results of 1.9mm cold-formed steel staggered hole bolted connections using M12-Bolts corresponding to failure process of the specimens A1.9-12-00 and A1.9-12-27

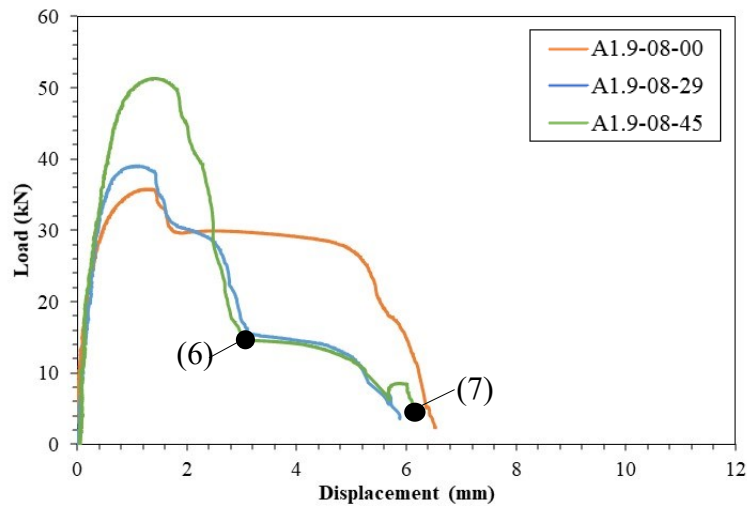


Figure 3.11: Test results of 1.9mm cold-formed steel staggered hole bolted connections using M8-Bolts corresponding to failure process of the specimen A1.9-08-45

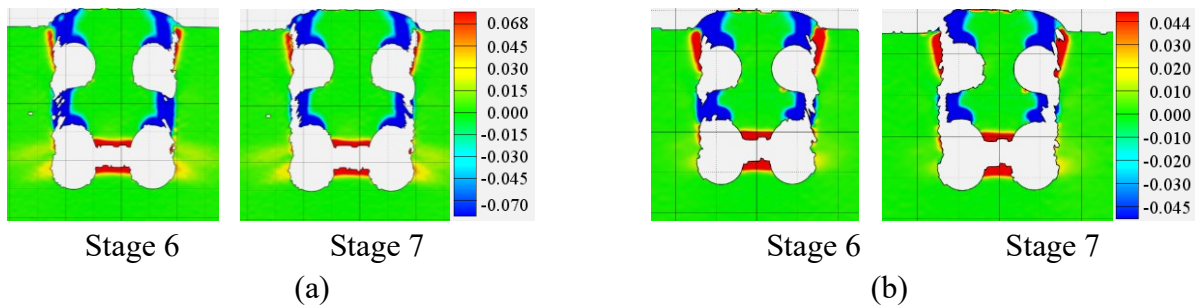


Figure 3.12: True strain distribution of the specimens (a) A1.9-12-00 and (b) A1.9-08-00 using DIC

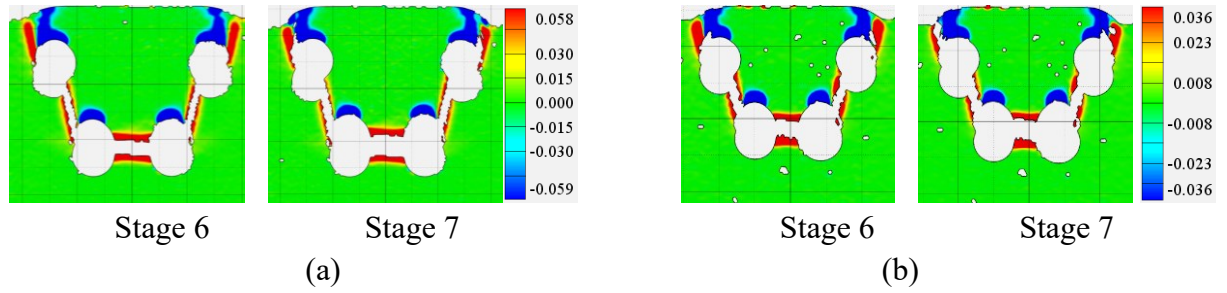


Figure 3.13: True strain distribution of the specimens (a) A1.9-12-27 and (b) A1.9-08-29 using DIC

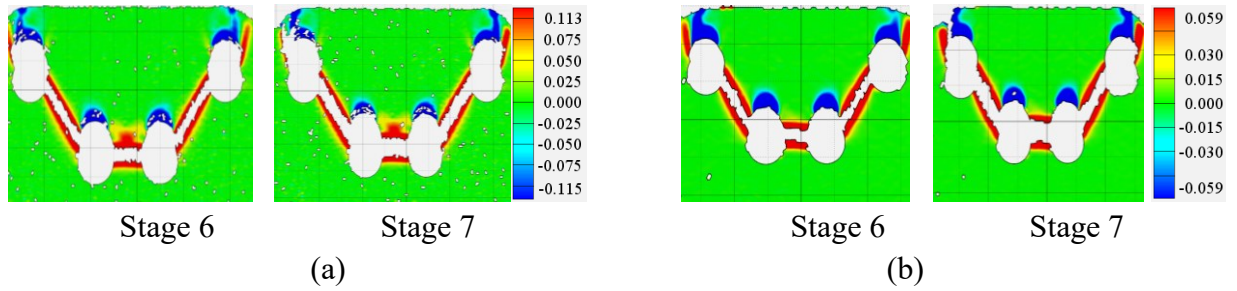


Figure 3.14: True strain distribution of the specimens (a) A1.9-12-45 and (b) A1.9-08-45 using DIC

3.3.3. Actual failure path in the inclined shear and tension zone

The full field of the displacement and the block shear failure mode of the staggered hole bolted connections were captured by means of Digital Image Correlation (DIC) during testing in conjunction with FEM modelling as illustrated in Figure 3.15. Consequently, the failure paths correlated closely with the angle α instead of occurring in the centerline between the staggered holes as they do for net section tension, as can be seen idealized in Figure 3.16a and experimentally in Figure 3.16b. The relationship between the length L' of the failure path relative to the actual centerline distance L and the angle α are demonstrated in detail in Figure 3.17 where both experimental and FEM values are used. Figure 3.17a is for large bolt holes and Figure 3.17b is for small bolt holes.

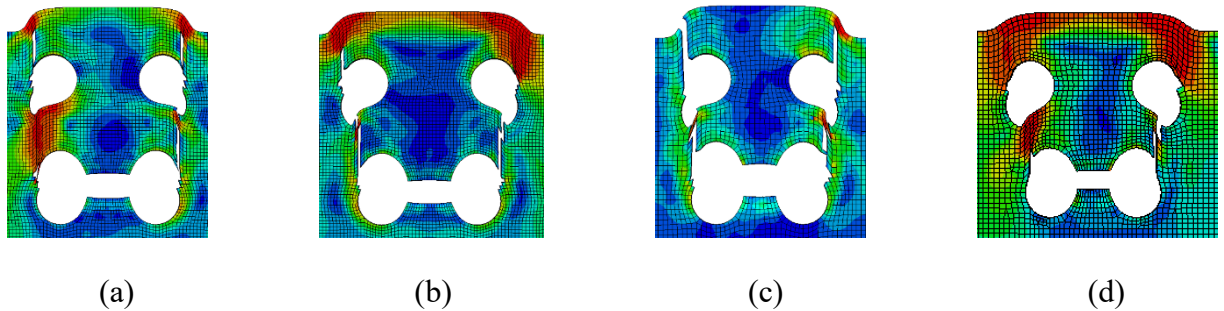
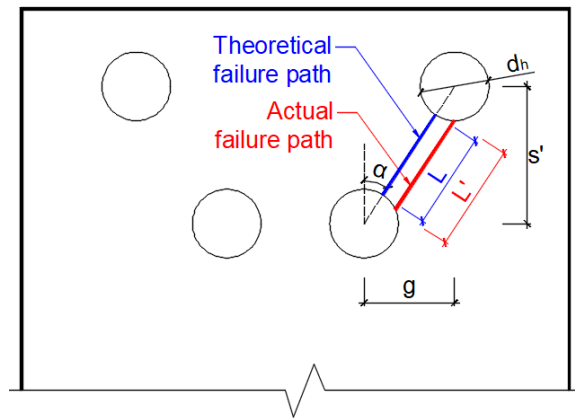
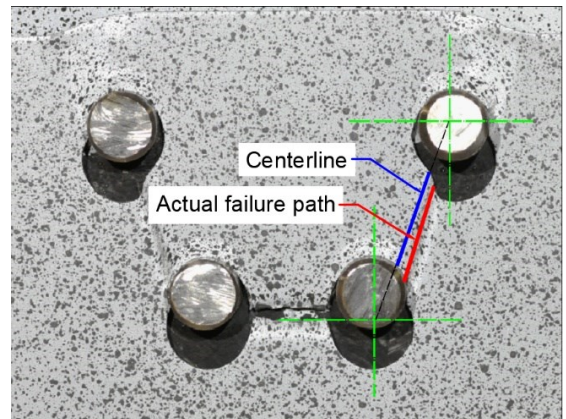


Figure 3.15: Von Mises strain distribution of (a) A2.4-12-04 specimen, (b) A2.4-12-10 specimen, (c) A2.4-08-06 specimen and (d) A2.4-08-13 using FEM modelling. Note: details of the specimens are demonstrated in Section 6.3.1.3.

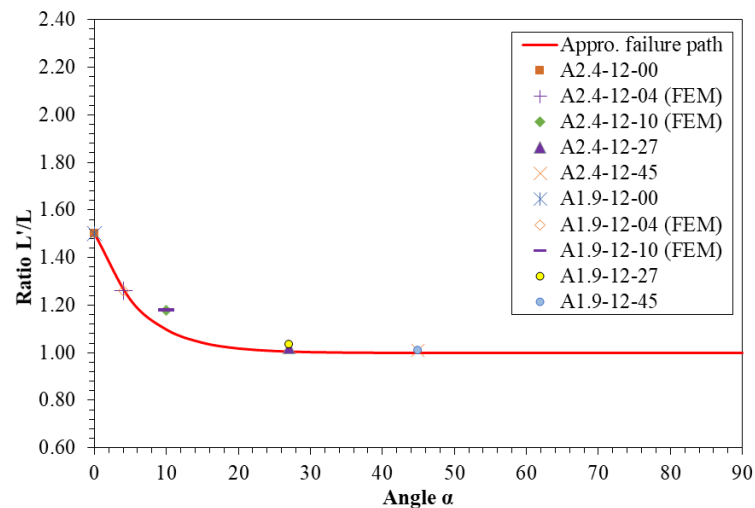


(a)

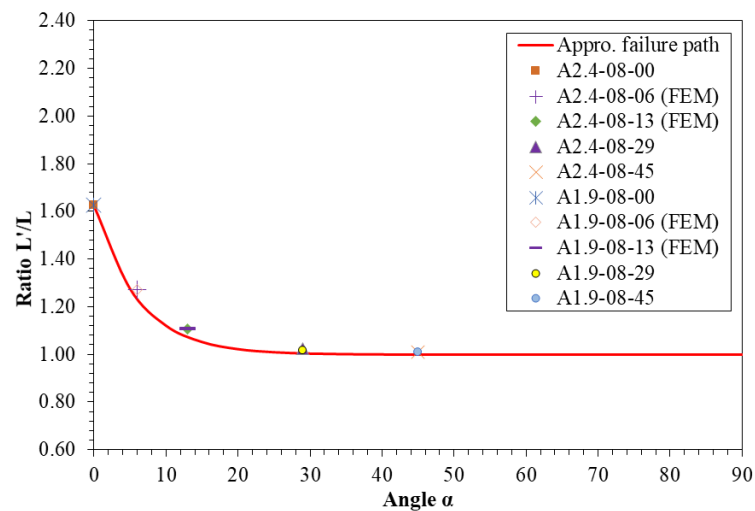


(b)

Figure 3.16: Actual failure path in a staggered hole bolted connection (a) in general and (b) of the specimen A2.4-12-27.



(a)



(b)

Figure 3.17: The relationship between the theoretical failure path and measured failure path of the staggered hole bolted connections using (a) M12-Bolts and (b) M8-Bolts.

Using the experimental behaviour demonstrated by the relationship between the theoretical failure path and measured failure path in Figure 3.17, an approximation to the failure path is proposed as

$$L' = L + \frac{d_h}{2} 10^{-4 \sin \alpha} = \sqrt{(s')^2 + g^2} - d_h + \frac{d_h}{2} 10^{-4 \sin \alpha} \quad (3.1)$$

3.3.4. Influence of the combined shear and tension ruptures on the inclined shear and tension zone

The experimented fracture behaviour observed in the fracture zones of the staggered hole bolted connections using a scanning electron microscope (SEM) for the specimens A2.4-12-00, A2.4-12-27 and A2.4-12-45 is illustrated in Figure 3.18 - Figure 3.20. The red arrows indicate the direction of captured surfaces, while the magnification ratio is further demonstrated in these figures through the ruler in the bottom right corner. The block shear rupture behaviour in the staggered paths correlated with the angle α . In particular, in the case of the specimen A2.4-12-00 (the angle $\alpha = 0^\circ$), a pure shear rupture occurred as in Figure 3.18. The fracture surface demonstrated a sliding failure between the two collapsed surfaces (due to shear band localization) without necking failure (due to growth of voids). By comparison, in the cases of the specimen A2.4-12-27 with $\alpha = 27^\circ$ shown in Figure 3.19 and the specimen A2.4-12-45 with $\alpha = 45^\circ$ shown in Figure 3.20a, necking failure appeared at the fracture surfaces due to the presence of tension rupture, similar to the necking failure behaviour in Figure 3.20b. Further, the influence of necking failure correlated with the angle α corresponding to the effect of shear rupture (i.e., Figure 3.19 & Figure 3.20a). These all demonstrated a combination of shear and tension ruptures in the inclined shear and tension zones of staggered hole bolted connections. Consequently, the nominal block shear strength in the staggered paths (i.e., R_n) includes two components which are the shear strength on the failure path parallel to the staggered path (i.e., R_n^s) and the tension strength on the failure path perpendicular to the staggered path (i.e., R_n^t), as demonstrated in Figure 3.21. These components can be added vectorially.

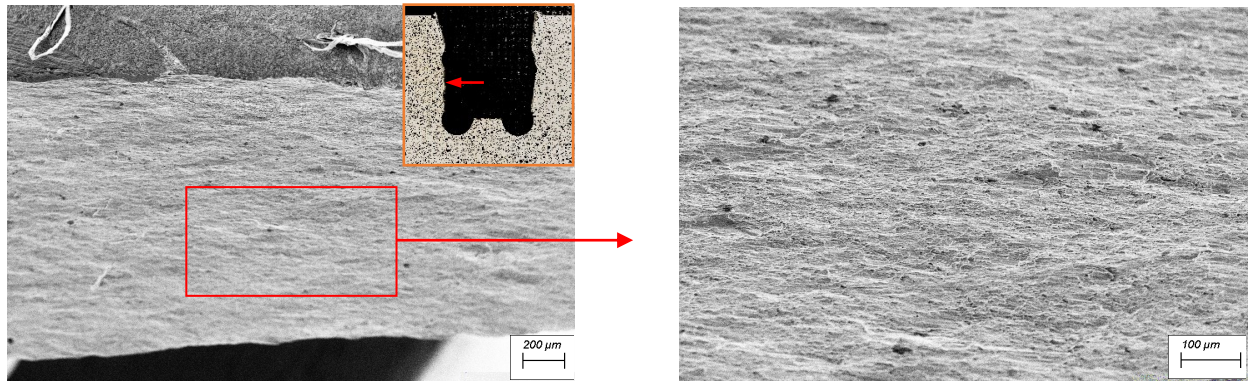


Figure 3.18: Schematic of staggered hole bolted connections failing in the inclined shear and tension zone of the specimen A2.4-12-00.

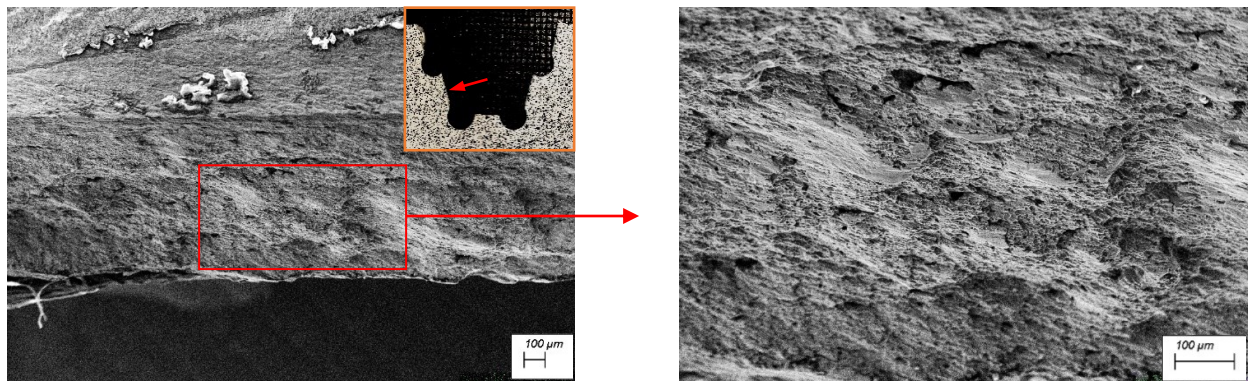


Figure 3.19: Schematic of staggered hole bolted connections failing in the inclined shear and tension zone of the specimen A2.4-12-27.

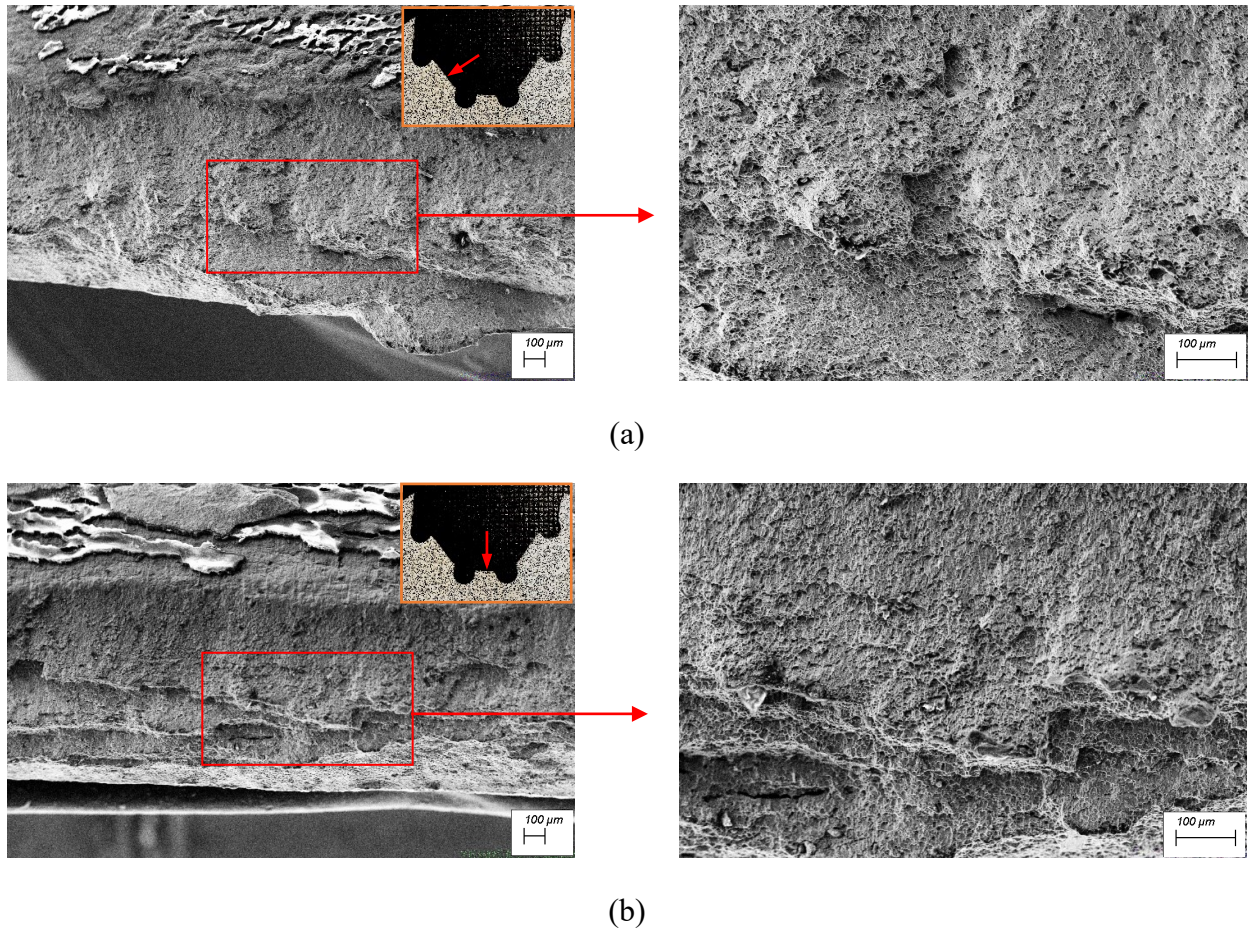


Figure 3.20: Schematic of staggered hole bolted connections failing in (a) the inclined shear and tension zone and (b) the pure tension zone of the specimen A2.4-12-45.

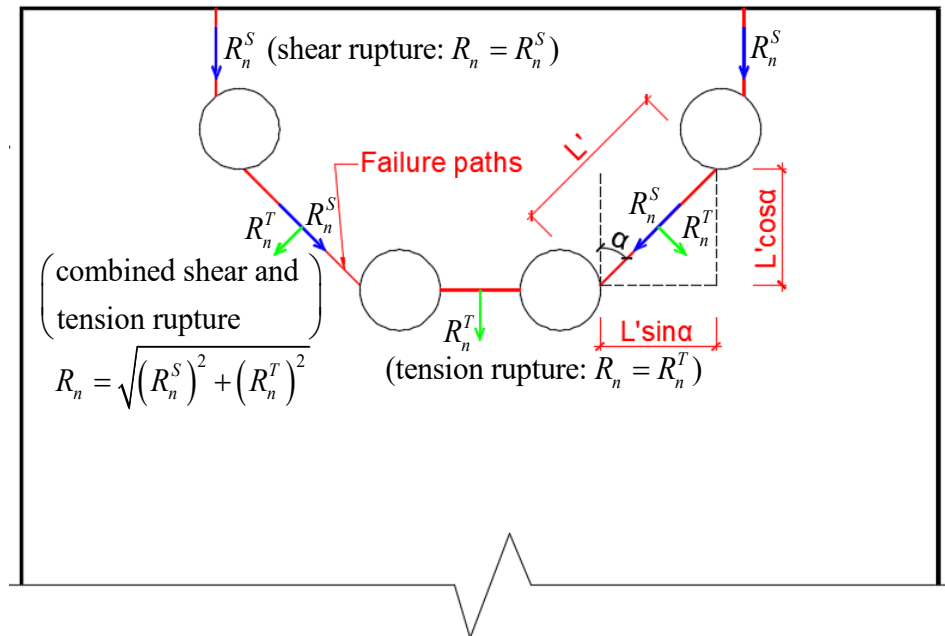


Figure 3.21: Shear and tension force at fracture zones.

Applying the principles in Teh and Clements [27] for block shear which have been incorporated in AS/NZS 4600:2018 [2] in conjunction with the above vectorial addition, a mechanics based formula is proposed as

$$R_n = 0.6f_y A_{av} + U_{bs} U_{sl} f_u A_{nt} + \sqrt{(R_n^S)^2 + (R_n^T)^2} \quad (3.2)$$

where $R_n^S = 0.6f_y tL' \cos \alpha$, and $R_n^T = f_u tL' \sin \alpha$. The components R_n^S and R_n^T are computed using L' defined by Equation (3.1); U_{bs} is a non-uniform block shear factor (0.5 for coped beam shear conditions with more than one vertical row of connectors, 1.0 for all other cases); and U_{sl} is the shear lag factor determined in AISI S100:16 (R2020) w/S3-22 - Section J6.2 [1]. Equation (3.2) has recently been balloted for acceptance in the 2024 Edition of AISI S100 (Ballot CS23-560B).

3.4. DESIGN STANDARD AND SPECIFICATION COMPARISONS

3.4.1. Current design equations

In the current cold-formed steel design standards and specifications from Australia and the USA, the block shear strength of conventional rectangular block shear bolted connections is specified. In particular, the nominal block shear rupture strength in AISI S100:16 (R2020) w/S3-22 - Section J6.3 [1] is determined as the lesser of the following

$$R_n = 0.6f_y A_{av} + U_{bs} U_{sl} f_u A_{nt} \quad (3.3)$$

where f_y and f_u represent yield stress, and tensile strength of the connected part, respectively; U_{bs} is a non-uniform block shear factor (0.5 for coped beam shear conditions with more than one vertical row of connectors, 1.0 for all other cases); U_{sl} is the shear lag factor; $A_{av} = (A_{gv} + A_{nv})/2$ is the active shear area lying in the middle of the net and gross shear areas. Gross area subject to shear, net area subject to shear, and net area subject to tension are denoted by A_{gv} , A_{nv} , and A_{nt} , respectively.

Further, according to the concepts in the Commentary on AISI S100:16-C (R2020) w/S3-22 - Section J6 [7], a method for determining the block shear strength without a consistent methodology for the inclined failure paths is proposed based on the Cochrane's formula

$$A_{nt} = n \left(g - d_h + \frac{(s')^2}{4g + 2d_h} \right) t \quad (3.4)$$

$$P_{nt} = f_u U_{sl} A_{nt} \quad (3.5)$$

where P_{nt} is the nominal tensile rupture strength; t is the thickness of steel sheet; d_h is the diameter of the hole; g is the gauge and s' is the stagger as shown in Figure 2.3. n is the number of gauges g .

The latest Joint Australian/New Zealand Standard – Cold-formed steel structures AS/NZS 4600:2018 [2] provides the nominal capacity for block shear rupture of the beam-end or tension member connection (R_n) shall be determined as follows

$$R_n = 0.6f_y A_{av} + f_u A_{nt} (0.9 + 0.1d_f/s_f) \quad (3.6)$$

in which A_{av} is the active shear area lying in the middle of the net and gross shear areas ($A_{av} = 2L_{av}t$); $L_{av} = L_{gv} - n_r d_h / 2$; L_{gv} is the distance from free edge to centreline of bolt furthest from the edge (Figure 2.2); n_r is the number of rows of bolts; the variable d_f denotes the bolt diameter; and s_f denotes the bolt spacing in the tensile resistance plane. The design Equation (3.6) does not include explicit equations for staggered hole block shear failure.

3.4.2. Proposed design equation

The accuracy of the proposed design Equation (3.2) is compared with the design standards and specifications from AISI S100:16 (R2020) w/S3-22 [1] and AS/NZS 4600:2018 [2] and the test results, as can be seen in Table 3.5. In particular, the ratio of test results divided by the predicted values ranges from 0.97 to 1.02 while that by AISI varies between 0.99 and 1.09. It is worth pointing out that the proposed formula is more accurate to obtain the nominal block shear strength in the cases of small angle α compared with the AISI due to the approximation of the inclined paths as tension paths in AISI design formulas. For instance, in the case of the specimen A2.4-12-27, the ratios of test result divided by the proposed value is only 1.00 while that by AISI is much higher at 1.09. Further, the same behaviour is observed for the other cases (i.e., A2.4-08-29, A1.9-12-27 and A1.9-08-29). The average of these ratios is 1.00 for proposed formula, and 1.05 for the AISI respectively. Therefore, the AISI predictions are more conservative in obtaining the nominal block shear strength against the test results in comparison with the proposed formula. In addition, the proposed design formula provides better results in comparison with AISI via the coefficient of variation, 1.57% and 3.37% respectively.

Table 3.5: Comparison of the proposed design Equation (3.2) with the test results and the design standards and specifications from AISI S100:16 (R2020) w/S3-22 [1] and AS/NZS 4600:2018 [2]

Specimen	Nominal thickness (mm)	Angle α (°)	Average test results P^T (kN)	Nominal block shear strength R_n (kN)			$\frac{P^T}{R_n^{AISI}}$	$\frac{P^T}{R_n^{Prop.}}$
				AISI S100:16 (R2020) w/S3-22 [1]	AS/NZS 4600:2018 [2]	Proposed formula		
A2.4-12-00	2.4	0	74.47	73.01	73.35	73.01	1.02	1.02
A2.4-12-27	2.4	27	79.03	72.81	-	79.40	1.09	1.00
A2.4-12-45	2.4	45	104.25	101.24	-	104.14	1.03	1.00
A2.4-08-00	2.4	0	46.70	46.01	46.34	46.01	1.01	1.01
A2.4-08-29	2.4	29	49.20	45.01	-	50.65	1.09	0.97
A2.4-08-45	2.4	45	65.14	62.08	-	64.90	1.05	1.00
A1.9-12-00	1.9	0	58.75	57.73	57.99	57.73	1.02	1.02
A1.9-12-27	1.9	27	62.58	57.67	-	62.83	1.09	1.00
A1.9-12-45	1.9	45	82.55	80.23	-	82.48	1.03	1.00
A1.9-08-00	1.9	0	36.01	36.37	36.63	36.37	0.99	0.99
A1.9-08-29	1.9	29	38.94	35.64	-	40.07	1.09	0.97
A1.9-08-45	1.9	45	51.04	49.18	-	51.39	1.04	0.99
Mean							1.05	1.00
C.o.V (%)							3.37	1.57

In addition, the proposed design Equation (3.2) is further compared in Table 3.6 with the independent test results on unconventional staggered block shear failures in Teh and Yazici [36] in conjunction with the specifications from AISI S100:16 (R2020) w/S3-22 [1] to verify its accuracy. Again, the proposed design Equation (3.2) provides better accuracy compared with the AISI with average normalized ratios of 0.98 and 0.94, respectively. The coefficients of variation of both the proposed design formula and the AISI are approximately similar, at 2.24% and 2.23% respectively.

Table 3.6: Comparison of the proposed design equation with the independent test results on unconventional staggered block shear failures in Teh and Yazici [36] and the specifications from AISI S100:16 (R2020) w/S3-22 [1]

Specimen	Nominal thickness (mm)	Angle α (°)	Test results in Teh and Yazici P^T (kN)	Nominal block shear strength R_n (kN)		$\frac{P^T}{R_n^{AISI}}$	$\frac{P^T}{R_n^{Prop}}$
				AISI S100:16 (R2020) w/S3-22 [1]	Proposed Formula		
CT55a*	3.0	53	109.20	115.75	111.72	0.94	0.98
CT55b*	3.0	53	109.20	115.75	111.72	0.94	0.98
CT56a*	3.0	53	100.80	105.49	102.34	0.96	0.98
CT56b*	3.0	53	98.50	105.49	102.34	0.93	0.96
CT57*	3.0	56	111.30	123.60	118.94	0.90	0.94
CT58*	3.0	56	103.90	113.33	109.56	0.92	0.95
CT65*	3.0	61	103.00	106.05	102.59	0.97	1.00
CT59*	3.0	61	111.10	116.31	111.97	0.96	0.99
CT60*	3.0	63	118.40	124.46	119.57	0.95	0.99
CT61*	3.0	68	112.70	117.71	113.09	0.96	1.00
Mean						0.94	0.98
C.o.V (%)						2.23	2.24

Note: *Details of the specimen configuration and material properties are given in Teh and Yazici [36]

3.4.3. Reliability analysis

In this section, the test results of the staggered hole bolted connections failing in the block shear mode are used for the reliability analysis of the proposed formula where the reliability indices (i.e., resistance factor ϕ and safety factor Ω) can be calibrated. Based on the reliability procedure employed in AISI S100:2016 (R2020) w/S3-22 [1], the basic concept of the LRFD method is established through the relationship between the design strength (ϕR_n) and the considered load combination ($\sum \gamma_i Q_i$), as can be seen in Equation (3.7)

$$\sum \gamma_i Q_i \leq \phi R_n \quad (3.7)$$

where γ_i is the load factor; Q_i denotes the load effect; R_n is the nominal resistance; and ϕ is the resistance factor.

In addition, the reliability index (β) is calculated as per Equation (3.8) where a greater value of β indicates a higher safety index and higher reliability in design.

$$\beta = \frac{\ln(R_m/Q_m)}{\sqrt{V_R^2 + V_Q^2}} \quad (3.8)$$

where R_m is the given mean resistance; Q_m is the mean load effect; V_R and V_Q are the corresponding coefficients of variation of the R_m and Q_m , respectively. The values of R_m , V_R and V_Q are respectively determined in Equations (3.9)-(3.11)

$$R_m = R_n (P_m M_m F_m) \quad (3.9)$$

$$V_R = \sqrt{C_P V_P^2 + V_M^2 + V_F^2} \quad (3.10)$$

$$V_Q = \frac{\sqrt{(D_m V_D)^2 + (L_m V_L)^2}}{(D_m + L_m)} \quad (3.11)$$

where C_P is the correction factor ($C_P = 5.7$ for $n_T = 3$ and $C_P = (1 + 1/n_T)(n_T - 1)/(n_T - 3)$ for $n_T \geq 4$); n_T denotes the number of tests; P_m is the mean ratio of the experimentally determined shear strength to the predicted shear strength; M_m is the mean ratio of the actual stress to the minimum specified stress of the steel material; F_m is the mean ratio of the actual thickness of the steel section to the specified value; V_P , V_M and V_F are the coefficients of variation of P_m , M_m and F_m , respectively. According to a report of BlueScope Steel for 1207 tests of CFS from 1.0 to 3.0mm summarised by Pham and Hancock [49], $M_m = 1.192$, $V_M = 0.031$, $F_m = 1.00$ and $V_F = 0.01$; D_m and L_m are the mean values of dead load and live load, respectively; V_D and V_L are the coefficients of variation of D_m and L_m , respectively.

Moreover, a gravity load including dead load and live load is designed for the connections in this research. According to AS/NZS 1170.0:2002 [50], load factors (1.2 and 1.5 for dead load and live load, respectively) are used for the typical combination of dead load-live load. Load statistics studies conducted by Ellingwood [51] suggested that $D_m = 1.05D$, $V_D = 0.10$, $L_m = L$ and $V_L = 0.25$.

Based on the given information above, Equation (3.8) can be rewritten as follows

$$\beta = \frac{\ln \left[\frac{1}{\phi} \frac{(1.2 D/L + 1.5)(P_m M_m F_m)}{(1.05 D/L + 1)} \right]}{\sqrt{C_P V_P^2 + V_M^2 + V_F^2 + \frac{(1.05 D/L)^2 0.1^2 + 0.25^2}{(1.05 D/L + 1)^2}}} \quad (3.12)$$

$$\text{or } \phi = \left(\frac{1.2 D/L + 1.5}{1.05 D/L + 1} \right) (P_m M_m F_m) \exp \left(-\beta \sqrt{C_P V_P^2 + V_M^2 + V_F^2 + V_Q^2} \right) \quad (3.13)$$

$$\text{and } \Omega = \frac{1}{\phi} \left(\frac{1.2 D/L + 1.5}{D/L + 1} \right) \quad (3.14)$$

In the above equations, the value of P_m is the mean value of the test-to-prediction ratio and the corresponding coefficient of variation (V_p) which were derived from the tests. Therefore, $P_m = 1.00$ and $V_p = 0.0157$ as can be seen in Table 3.5.

Figure 3.22 demonstrates the relationship between the reliability index (β) and the ratio of dead load to live load (D/L) for various values of the resistance factor (ϕ). In particular, a lower value of the resistance factor (ϕ) provides a higher index of the reliability (β). Nevertheless, a higher value of the reliability index (β) can be obtained when using a higher value of the ratio of (D/L). For the purposes of checking the reliability based on LRFD criteria, the ratio D/L is assumed as 0.2. This ratio is typical for light weight CFS structures. Furthermore, the target reliability index (β) is taken as 3.5 for CFS connections for LRFD per AISI S100:2016 (R2020) w/S3-22 – Section K2.1 [1] so that failure of a structure is less likely to be initiated in the connections. Consequently, the resistance factors (ϕ) and safety factors (Ω) of the proposed formula for rupture of the bolted connections with $D/L = 0.2$ and $\beta = 3.5$ is illustrated in Table 3.7. These reliability indices (i.e., $\phi = 0.82$ and $\Omega = 1.77$) reflect a good quality control of the materials involved in the tests and the accuracy of the proposed formula in comparison with the CFS Specification from AISI S100:2016 (R2020) w/S3-22 [1] (i.e., $\phi = 0.75$ and $\Omega = 2.00$).

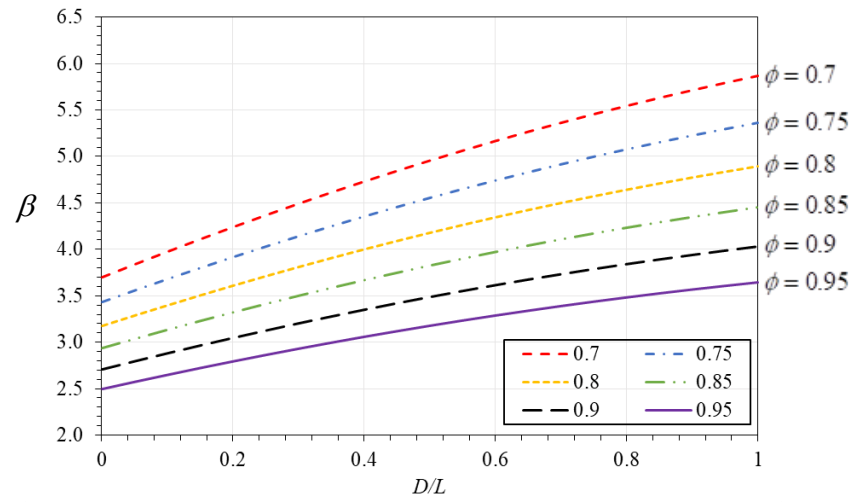


Figure 3.22: Reliability index β with ratio of dead load to live load D/L for different values of the resistance factor ϕ

Table 3.7: Resistance factor ϕ and safety factor Ω of the proposed formula with $D/L = 0.2$ and $\beta = 3.5$

Information		Resistance factor ϕ	Safety factor Ω
Mean	$P_m = 1.00$		
C.o.V	$V_p = 0.0157$	0.82 (0.75)*	1.77 (2.00)*
Number of samples	$n_T = 12$		

Note: * Values obtained from the AISI S100:2016 (R2020) w/S3-22 [1] Specification

3.5. SUMMARY

A total of 24 tests using G450 sheets were conducted under quasi-static monotonic loading to investigate the complex behaviour of the staggered hole bolted connections failing in a block shear mode. The inclined shear and tension zone between the staggered holes not only increased the block shear strength but also changed the whole behaviour of the bolted connections. In particular, the failure states involving pure tension, inclined shear and tension and the remaining pure shear failure were different because of the effect of the staggered paths. Furthermore, the presence of the staggered holes also reduced the ductility of the bolted connections in comparison with traditional rectangular block shear connections.

The full field of the displacement and the block shear failure mode of the connections were captured by means of Digital Image Correlation (DIC) during the testing. Consequently, the fracture paths in the inclined shear and tension zone were shown to correlate clearly with the angle α instead of occurring in the centerline between the staggered holes.

The nominal shear strength of the staggered hole bolted connections with staggered paths correlated with the angle α by using the scanning electron microscope (SEM) technique. The combination of shear and tension ruptures can be accounted by two components which are the shear strength on the failure path parallel to the staggered path and the tension strength on the failure path perpendicular to the staggered path. As a result, a design equation for block shear strength with staggered holes is proposed, which provides a better correlation with the experimental results than current design standards. In addition, the AISI S100 predictions are more conservative in obtaining the nominal block shear strength against the test results in comparison with the proposed formula while AS/NZS 4600 does not include specific equations for staggered block shear failures.

With a target reliability index (β) of 3.5, the reliability analysis demonstrates that the reliability indices (i.e., resistance factor $\phi = 0.82$ and safety factor $\Omega = 1.77$) are better than the current values used for block shear rupture of the CFS staggered hole bolted connections.

CHAPTER 4

BEHAVIOUR ON HIGH STRENGTH SHEAR BEAM-END BOLTED CONNECTIONS IN COLD-FORMED STEEL CHANNELS

4.1. INTRODUCTION

A shear beam-end bolted connection failing in block shear is investigated in this chapter where the cold-formed steel (CFS) channels are bolted to mild steel T-shaped sections which are then fixed to a tubular column. A dual-actuator setup comprising of two actuators is used in the test apparatus to investigate the effect of the combined shear force and beam-end rotation on the block shear behaviour and shear strength of the connections. The shear distance between the bolts is the main variable to evaluate the effect of the shear fracture paths and fracture process behaviour on the shear rupture strength of the connections. In addition, the parameters such as the rotation at the connections, the thicknesses of the CFS channels, the edge distances and the bolt sizes are also studied in this chapter. Consequently, the test results are compared to the current cold-formed steel design standards and specifications from AS/NZS 4600:2018 [2] and AISI S100:2016 (R2020) w/S3-22 [1] to verify whether existing design equations and methodologies are accurate for determining the shear capacity of the shear beam-end bolted connections under the combined effects of shear-rotation and fracture process behaviour. A reliability analysis is included in this chapter.

4.2. EXPERIMENTAL INVESTIGATION OF SIMPLE SHEAR BOLTED CONNECTIONS

The experimental program was performed in the JW Roderick Structures Laboratory in the School of Civil Engineering, the University of Sydney using a dual-actuator connection testing machine. In addition, the AISI S905-13: Test Standard for Cold-formed Steel Connections [8], Specification for Structural Joints Using ASTM A325 or A490 Bolts [9], Research report RP01-4 – AISI: Testing of Bolted Cold-Formed Steel Connections in Bearing (With and Without Washers) [10], and the European Recommendations – ECCS TC7 TWG 7.10: The Testing of Connections with Mechanical Fasteners in Steel Sheeting and Sections [11] were employed as the testing methodologies.

Cold-formed G450 and G500 steel channels to AS 1397:2021 [52] were used for the specimens to obtain a range of high strength material properties. Average material properties of the CFS channels were obtained from coupon tests taking in the longitudinal direction (Figure 4.1), as can be seen in Table 4.1.

A typical test arrangement of the shear bolted connections using two MTS hydraulic actuators is demonstrated in Figure 4.2 and Figure 4.3. One actuator was located adjacent to the connections (i.e., Actuator B) to apply the shear force while the other actuator was located at the free end of the cantilever beam (i.e., Actuator A) to produce the rotation. Each actuator had a maximum capacity of 253kN in compression and 162kN in tension. The CFS channel was connected to the tubular column through a mild steel T-shaped section (thickness of 10.0mm). High strength steel Grade 8.8 bolts were used to connect the CFS channel to the mild steel T-shaped section as well as to eliminate shear failure in the bolts. The flanges of the T-shaped section were bolted to the tubular column by four high strength steel Grade 8.8 M16 bolts. Furthermore, two different diameters of the bolts (i.e., 8mm and 12mm) were employed in the tests to investigate the hole size effect on the shear fracture lines and the capacity of the connections. The major failure mode in this series was in-plane block shear failure while all bolts were hand-tightened only. Therefore, effects of washers (i.e., M8×16mm×1.2mm stainless A2-70 G304 flat washers for M8 bolts and M12×24mm×2.5mm stainless A2-70 G304 flat washers for M12 bolts) on the test results were negligible. The centre-to-centre spacing of the bolts was changed to investigate the influence of the shear fracture lines on the shear rupture strength of the connections. The minimum distance from the centre of the bolts to the edge of the steel plate was 15 mm which satisfied the minimum requirement for edge distance stipulated by the AS 4100:2020 [53] and the AISI S100:2016 (R2020) w/S3-22 [1].

Moreover, the web of the CFS channel was strengthened by a mild steel stiffener using G14 screws (diameter of 6.3mm) to eliminate buckling failure in the web. When using the short stiffener (thickness of 6.0mm) as demonstrated in Figure 4.4b, shear buckling behaviour in the CFS channel occurred caused by the in-plane bending. In order to isolate the investigated failure mode in this research (i.e., block shear failure), a thicker web stiffener (thickness of 10.0mm) was utilised (see Figure 4.5). In particular, the new stiffener was extended to the connections to eliminate the shear buckling behaviour occurring in the CFS channel. Therefore, the new stiffener was chosen as the ideal web stiffener and was also employed throughout for all specimens investigated.

Table 4.1: Average material properties of the CFS channels (full test data in Appendix B)

Steel	Nominal thickness (mm)	Base metal measured thickness (mm)	Yield stress f_y (MPa)	Ultimate strength f_u (MPa)
G500	1.20	1.22	632.10	645.27
G450	1.50	1.52	581.17	616.61

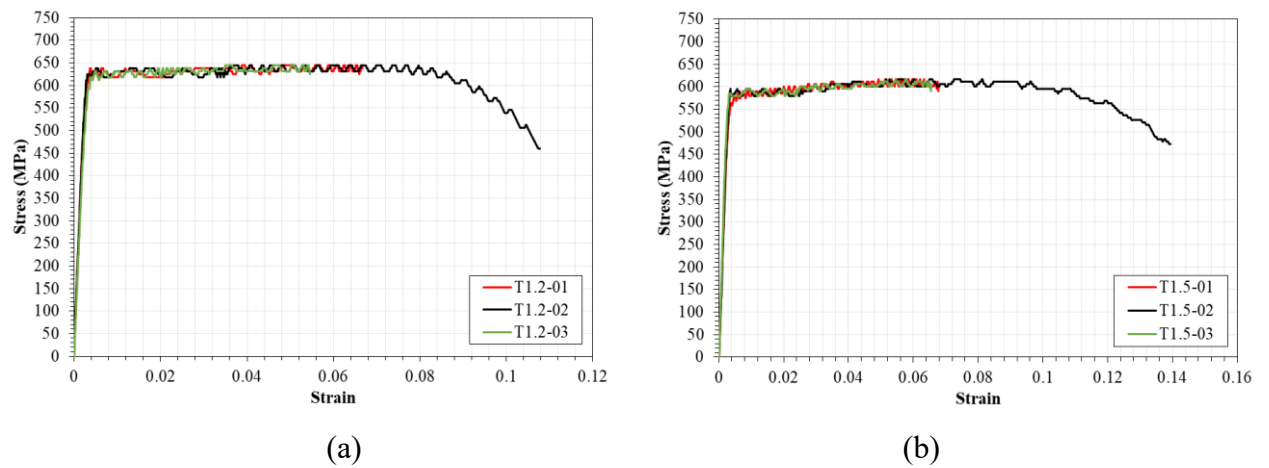


Figure 4.1: Stress-strain curves of (a) 1.2mm G500 cold-formed sheets and (b) 1.5mm G450 cold-formed sheets

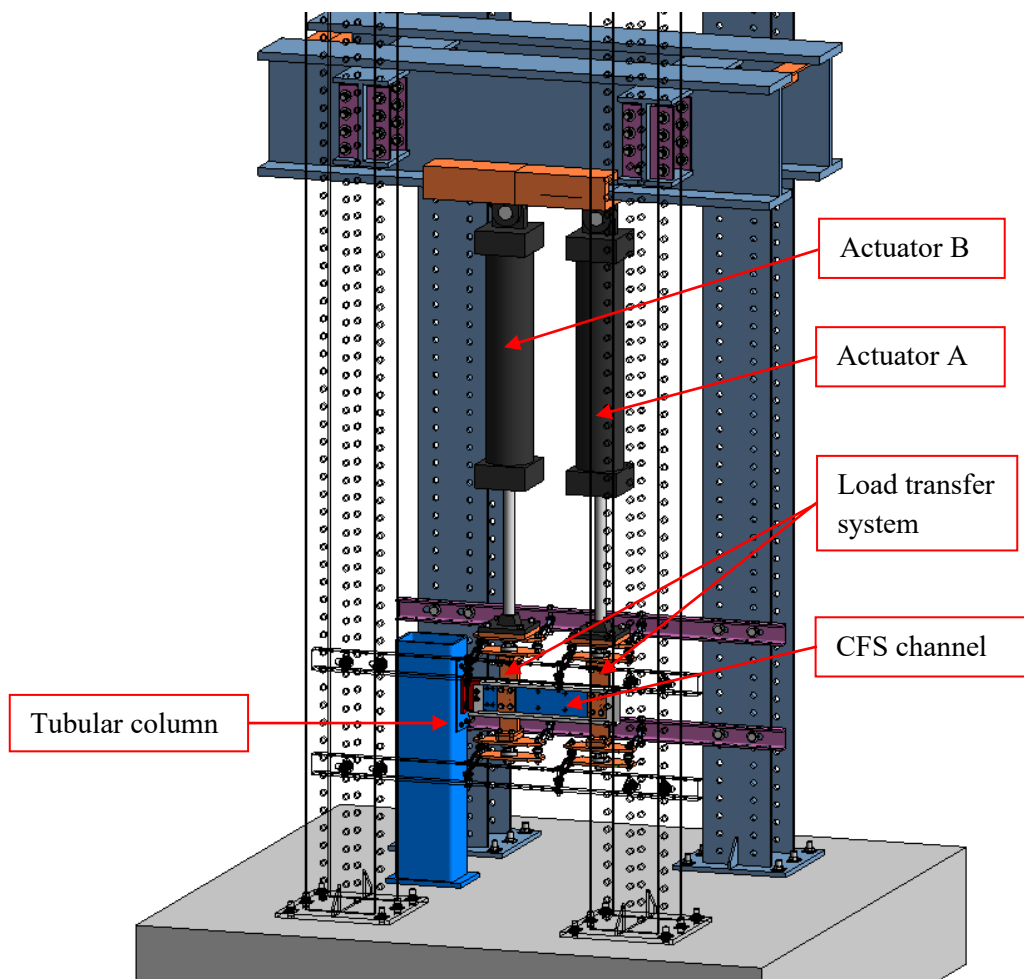


Figure 4.2: Elevation view of a typical test setup

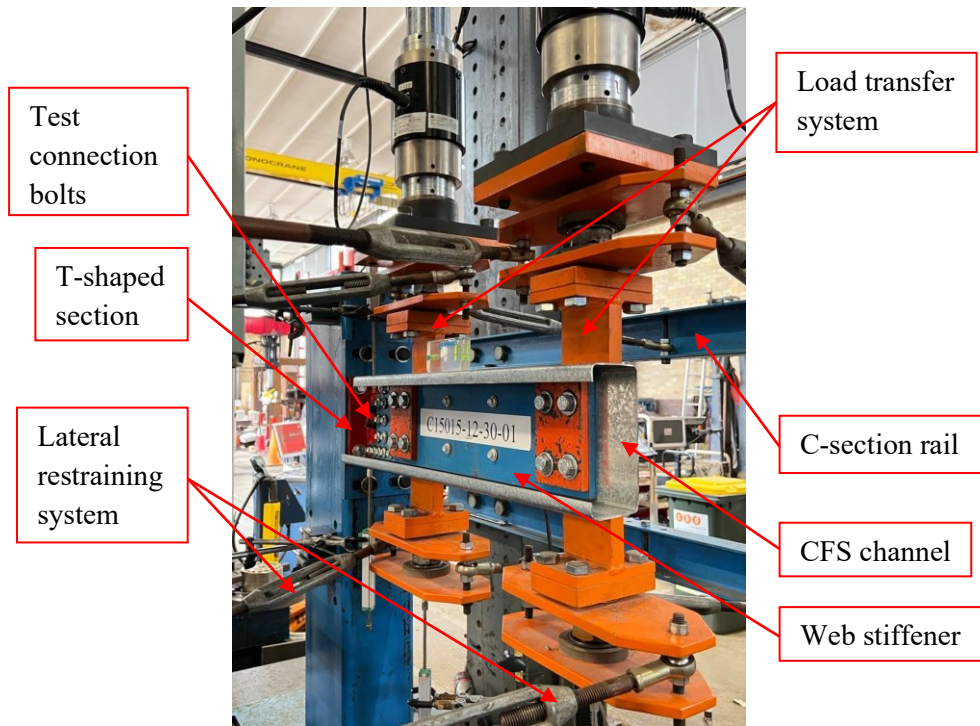


Figure 4.3: Connection details

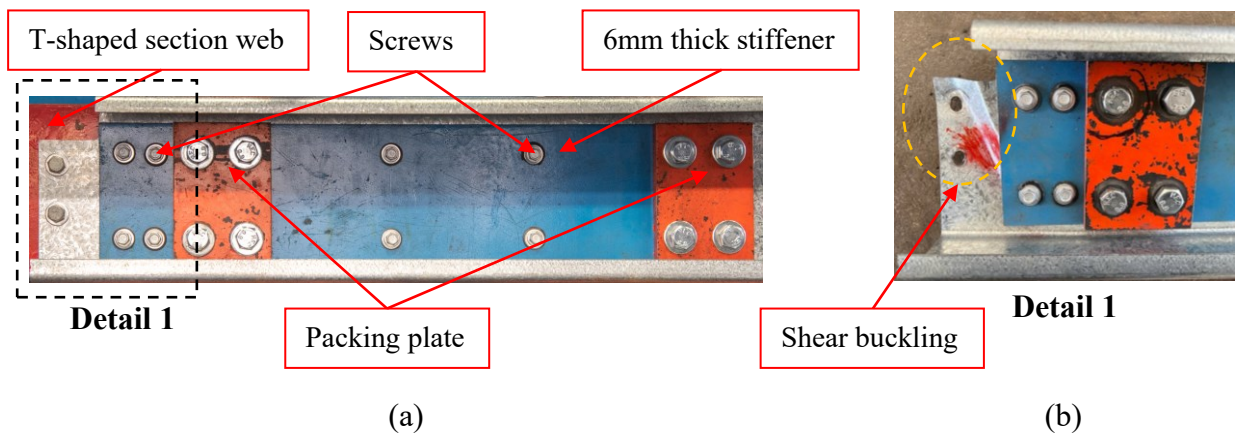


Figure 4.4: Shear buckling behaviour of the CFS channel using the short stiffener (thickness of 6.0mm)

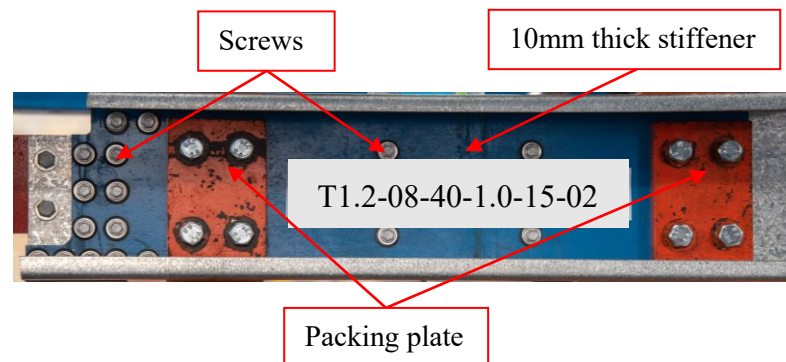


Figure 4.5: Detail of the ideal web stiffener (thickness of 10.0mm)

Vertical forces induced by the dual actuator were transferred to the CFS channels through a load transfer system as illustrated in Figure 4.6. The I-section beam of the system was connected to the CFS channel through packing plates in conjunction with circular spacers. These circular spacers played a crucial role to enable the I-section beam to rotate around the steel shank without creating any restrained force at the joints under investigation. Furthermore, three spacers were placed between the back of the channel and the I-section beam so that the force was applied at the shear centre of the channel to minimise the effect of torsion.

The load transfer system was restrained against lateral displacement by four lateral restraining systems located at the top and bottom of each I-section beam, as can be seen in Figure 4.7. Each system included two threaded bars which were connected to each other by a turnbuckle to form a bracing rod whose length was adjustable by twisting the turnbuckle. The length of each bracing rod was checked and adjusted during the experiments to keep the channel always vertical so that the channel was only subjected to in-plane shear force and moment. The free ends of the bracing rod were connected to a pivot plate or a C-section rail by a swivel ball joint bearing which allowed vertical displacement and in-plane rotation when the channel moved downwards.

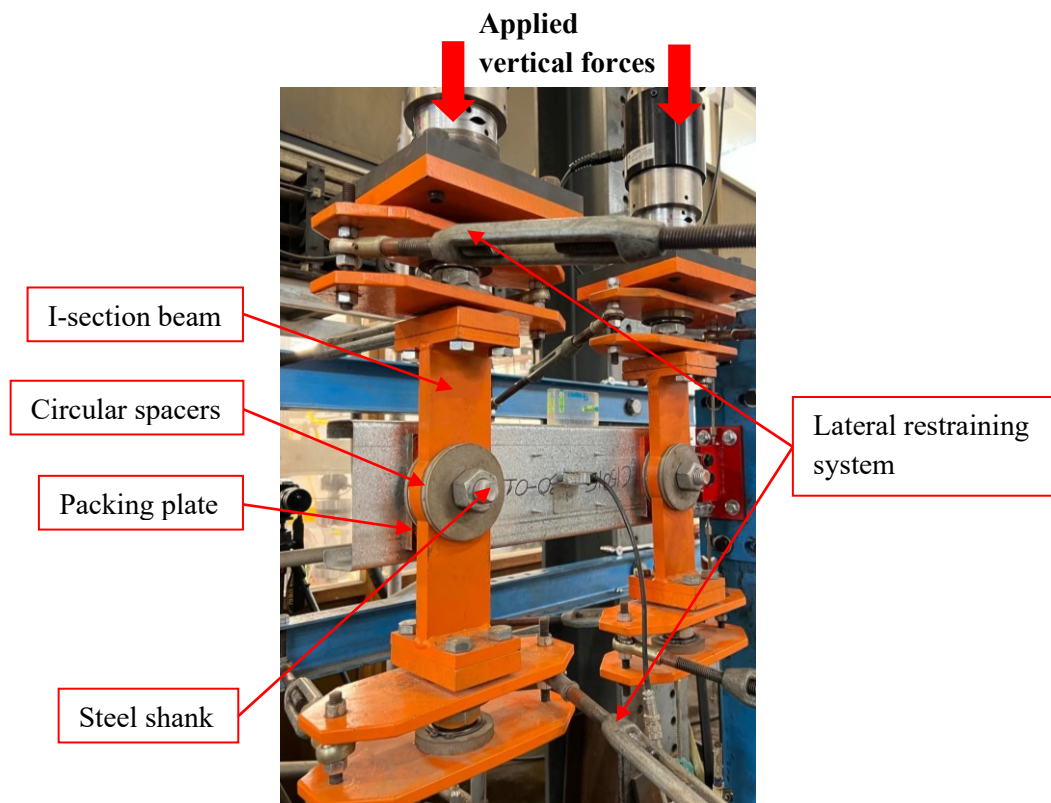


Figure 4.6: Load transfer system

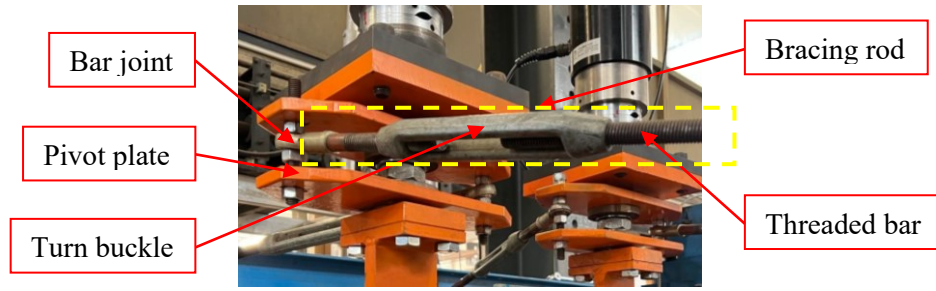


Figure 4.7: Lateral restraining system

Load results from the MTS hydraulic actuators and the displacements from the linear variable displacement transducers (LVDTs) in conjunction with the rotation from the inclinometers were recorded simultaneously by a data logger (Figure 4.8). In particular, one inclinometer (INCL-1) was used to determine the in-plane rotation of the channel at the mid-point of the loading locations whereas four linear variable displacement transformers (LVDTs) were used to determine the vertical displacement at the connection (LVDT-1), the vertical slip of the T-shaped section (LVDT-2), and the vertical displacements at the loading locations (LVDT-3 and LVDT-4). In addition, the speed of each actuator was predetermined and fixed throughout the experiment. The movement speed of Actuator B was consistently slow at the rate of 0.5 mm/min while the running ratio (RR) (the ratio of speed of Actuator A over the speed of Actuator B) varied between 1.0, 2.0 or 3.0 to create different rotations at the connections. These movement speeds were used to ensure a quasi-static response. At least two tests were repeated for each type of configuration. The geometric details of the test specimens are given in Table 4.2 and Figure 4.9. In particular, the CFS channels used in the experiments were C15012 and C15015 lipped channels with respectively thicknesses (t) of 1.2mm and 1.5mm. The CFS channels were made from high-strength cold-reduced sheet steel to AS 1397:2021 [52] whereas the T-shaped sections were made from hot-rolled steel plates to AS/NZS 3678:2016 [54]. The tubular column was Grade C450L0 RHS250x150x12 to AS/NZS 1163:2016 [55]. All the bolts were high-strength steel Grade 8.8 bolts to AS/NZS 1252.1:2016 [56].

Two main geometric variables shown in Figure 4.9 investigated in this research were the shear distance s (i.e., $s = 30, 40, 50\text{mm}$ for M12-bolts and $s = 20, 30, 40\text{mm}$ for M8-bolts) and the edge distance e_2 (i.e., $e_2 = 20, 25, 30\text{mm}$ for M12-bolts and $e_2 = 15, 20, 25\text{mm}$ for M8-bolts) to investigate respectively the effect of the shear fracture paths and fracture process behaviour on the shear rupture strength of the connections. Nevertheless, the end distance e_1 remained unchanged (i.e., $e_1 = 25\text{mm}$ for M12-bolts and $e_1 = 20\text{mm}$ for M8-bolts) to eliminate the out-of-plane curling phenomenon when using higher end distances.

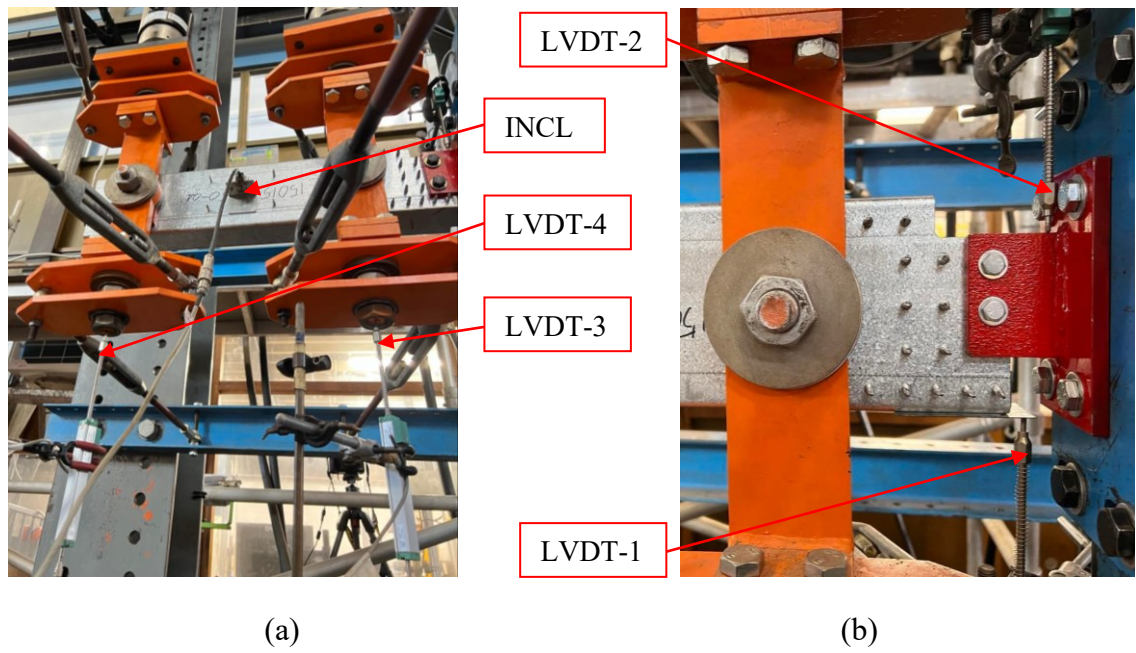


Figure 4.8: Linear variable displacement transducers and inclinometer positions

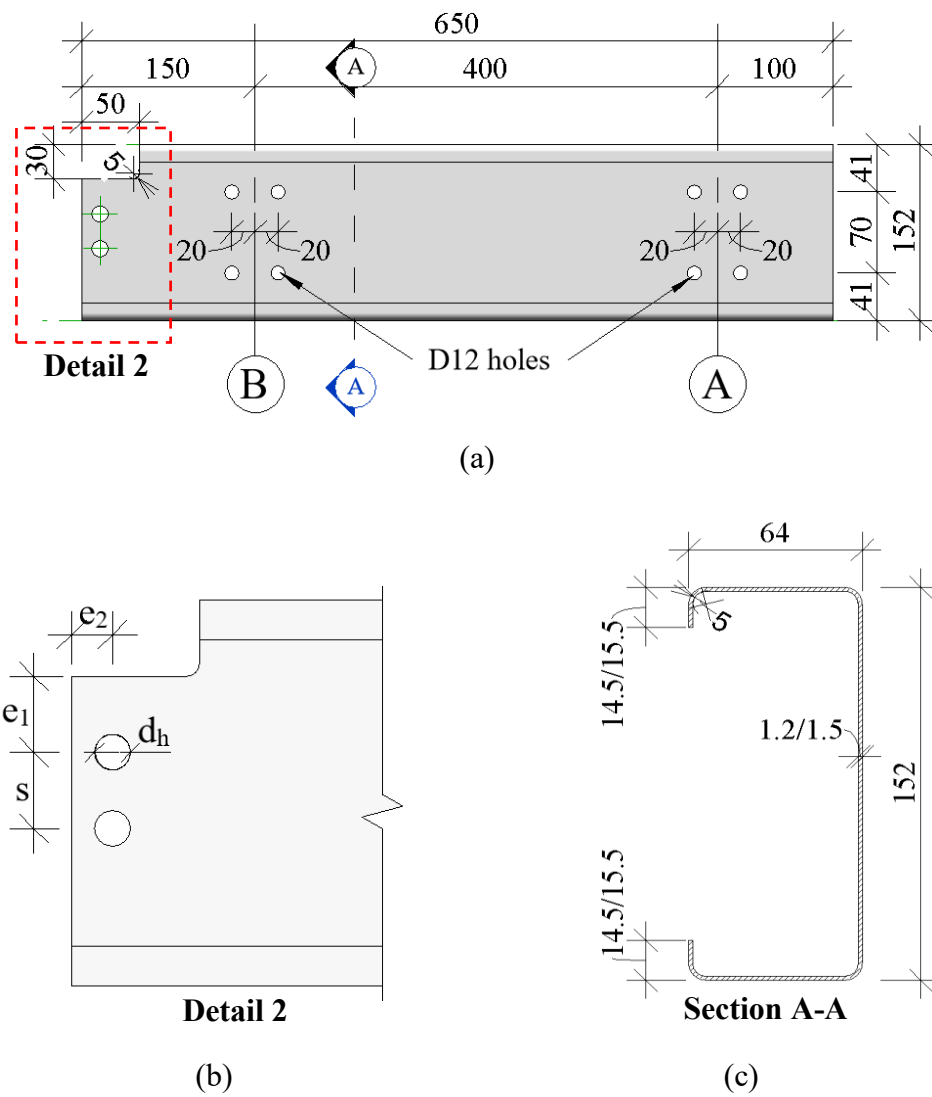


Figure 4.9: Schematic of the CFS channels configurations

Table 4.2: Specimen configurations

Specimen	Nominal thickness t (mm)	Nominal hole size d_h (mm)	Nominal bolt size d_f (mm)	End distance e_1 (mm)	Distance s (mm)	Edge distance e_2 (mm)
T1.2-12-30-RR-20	1.20	14	12	25	30	20
T1.2-12-40-RR-20	1.20	14	12	25	40	20
T1.2-12-50-RR-20	1.20	14	12	25	50	20
T1.2-12-30-RR-25	1.20	14	12	25	30	25
T1.2-12-40-RR-25	1.20	14	12	25	40	25
T1.2-12-50-RR-25	1.20	14	12	25	50	25
T1.2-12-30-RR-30	1.20	14	12	25	30	30
T1.2-12-40-RR-30	1.20	14	12	25	40	30
T1.2-12-50-RR-30	1.20	14	12	25	50	30
T1.2-08-20-RR-15	1.20	09	8	20	20	15
T1.2-08-30-RR-15	1.20	09	8	20	30	15
T1.2-08-40-RR-15	1.20	09	8	20	40	15
T1.2-08-20-RR-20	1.20	09	8	20	20	20
T1.2-08-30-RR-20	1.20	09	8	20	30	20
T1.2-08-40-RR-20	1.20	09	8	20	40	20
T1.2-08-20-RR-25	1.20	09	8	20	20	25
T1.2-08-30-RR-25	1.20	09	8	20	30	25
T1.5-12-30-RR-20	1.50	14	12	25	30	20
T1.5-12-40-RR-20	1.50	14	12	25	40	20
T1.5-12-50-RR-20	1.50	14	12	25	50	20
T1.5-12-30-RR-25	1.50	14	12	25	30	25
T1.5-12-40-RR-25	1.50	14	12	25	40	25
T1.5-12-50-RR-25	1.50	14	12	25	50	25
T1.5-12-30-RR-30	1.50	14	12	25	30	30
T1.5-12-40-RR-30	1.50	14	12	25	40	30
T1.5-12-50-RR-30	1.50	14	12	25	50	30
T1.5-08-20-RR-15	1.50	09	8	20	20	15
T1.5-08-30-RR-15	1.50	09	8	20	30	15
T1.5-08-40-RR-15	1.50	09	8	20	40	15
T1.5-08-20-RR-20	1.50	09	8	20	20	20
T1.5-08-30-RR-20	1.50	09	8	20	30	20
T1.5-08-40-RR-20	1.50	09	8	20	40	20
T1.5-08-20-RR-25	1.50	09	8	20	20	25

Note: RR – Running ratio

4.3. EXPERIMENTAL RESULTS

This section describes the experimental results of the shear bolted connections with different thicknesses, hole sizes, running ratios and shear distances between the two holes to investigate the influence of the shear fracture lines and combined effects of shear-rotation on the shear rupture strength and fracture behaviour of the connections. Two tests were repeated for each type of configuration. The dimensions are similar for the repeated tests, in which their test names have a suffix added (i.e., 01, 02, etc.). For example, all the test names T1.2-12-30-RR-20-01, T1.2-12-30-RR-20-02 have a similar configuration.

4.3.1. Running ratio $RR=1.0$

4.3.1.1 Shear bolted connections tests of 1.2mm cold-formed steel channels

Table 4.3 provides the test results of 1.2mm cold-formed steel shear bolted connections where the running ratio ($RR=1.0$) was employed. It is noticeable that all test results are consistent. Specimens were fabricated with thickness of 1.2mm and the shear distances (s) between the two bolts was the main variable. Therefore, the shear distance s not only significantly affected the shear strength but also changed the fracture process of the shear bolted connections. In particular, the shear strength of the connections using the M12-high strength steel bolts with $e_2 = 20mm$ increased approximately by 9% when the shear distance s increased from 30mm to 40mm. Likewise, similar improvement was observed as s increased from 40mm to 50mm. Similar behaviour was observed for the remaining cases with $e_2 = 25mm$ and 30mm illustrating the minor effect of the edge distance e_2 on the block shear behaviour of the shear beam-end bolted connections.

Furthermore, a large improvement also occurred in the case of the shear bolted connections using the smaller M8-high strength steel bolts with $e_2 = 15mm$ although the change between the shear distances was similar as for the previous case (i.e., M12-high strength steel bolts with $e_2 = 20mm$). In this configuration, the shear strength improved nearly 13-14% from $s = 20mm$ to $s = 30mm$ and from $s = 30mm$ to $s = 40mm$. Again, similar behaviour occurred in the remaining cases of the shear bolted connections using the M8-bolts with $e_2 = 20mm$ and 25mm. However, in the cases of bolted connections with both the edge distance e_2 and shear distance s relatively long (i.e., specimens T1.2-08-40-1.0-20-01, T1.2-08-40-1.0-20-02, T1.2-08-30-1.0-25-01 and T1.2-08-30-1.0-25-02), bearing failures occurred which was not of interest to investigate further in these configurations.

The load-displacement behaviour of the shear bolted connections using M8-bolts with $e_2 = 15\text{mm}$ is demonstrated in Figure 4.10a. Although all three configurations illustrate pure block shear rupture, the fracture process and ultimate load significantly correlate to the shear distance s . The relationship between the load and displacement with $s = 40\text{mm}$ is discretized into six stages in Figure 4.10a & Figure 4.10b. The connection in this case provided the highest shear strength and ductility. In the beginning, the connection experienced a slight slip without strength improvement because of the tolerance between bolt size and hole size (e.g., Stage 0). After that the shear strength of the connection increased gradually in the elastic phase (e.g., Stage 1) before transforming to the hardening range (e.g., Stage 2). In this stage, yielding started to occur in the tension zone at the bottom bolt because this region is more critical for the strain distribution in comparison with the shear zones. After the shear strength reached the peak (e.g., Stage 3), tensile fracture appeared at the bottom bolt (e.g., Stage 4) which caused a sudden strength reduction on the load-displacement curve. At this stage, yielding also started to occur in the shear zones until shear fracture appeared gradually between the two bolts in Stage 5, followed by a total shear fracture in Stage 6. Clearly, in this configuration, the shear fracture between the bolts occurred later than the tensile fracture at the bottom bolt by using the relatively long shear distance (s).

Similar pure block shear fractures were observed in the remaining cases with $s = 20\text{mm}$ and 30mm . Nevertheless, after the tensile fracture occurred at the bottom bolt, the shear fracture between the two bolts also appeared simultaneously when using the relatively short shear distances (s). Furthermore, the fracture behaviour of these configurations is shown in Figure 4.11 where the active shear resistance planes as proposed by Teh and Clements [27] is confirmed. Further, the load-displacement curves of the bolted connections using M8-bolts with longer edge distance (i.e., $e_2 = 20\text{mm}$) are shown in Figure 4.12 where bearing failure is demonstrated through a slight strength reduction on the load-displacement curve of the specimen T1.2-08-40-1.0-20-02 in conjunction with the fracture behaviour in Figure 4.13.

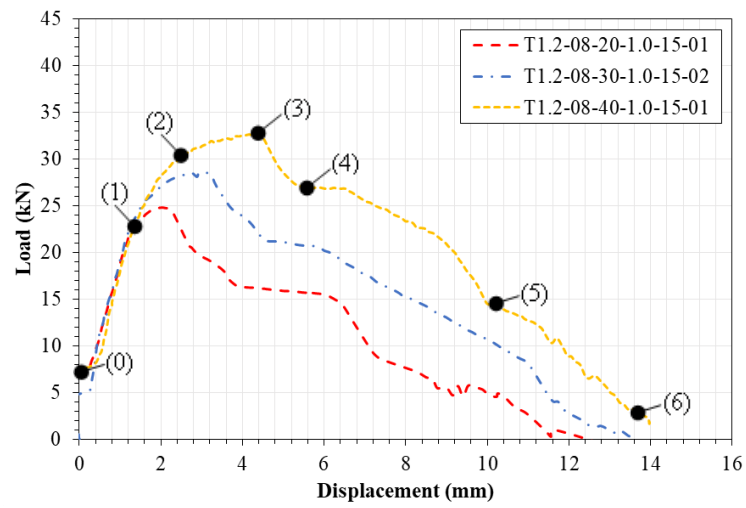
The behaviour of these types of configurations were further studied using the M12-high strength steel bolts with $e_2 = 20\text{mm}$ to investigate the effect of hole size on the shear bolted connections. It is apparent from Figure 4.14 that the behaviour of pure block shear rupture of these configurations has the same trend as that observed in Figure 4.10a. By comparison, the block shear strength of the larger hole connections was higher by nearly 30% than that of the smaller hole connections though both types of connections had similar ductility. The fracture behaviour of the larger hole connections is further demonstrated in Figure 4.15, which confirms

the active shear resistance planes between the bolts. Details of full set of load-displacement curves are given in Appendix C.

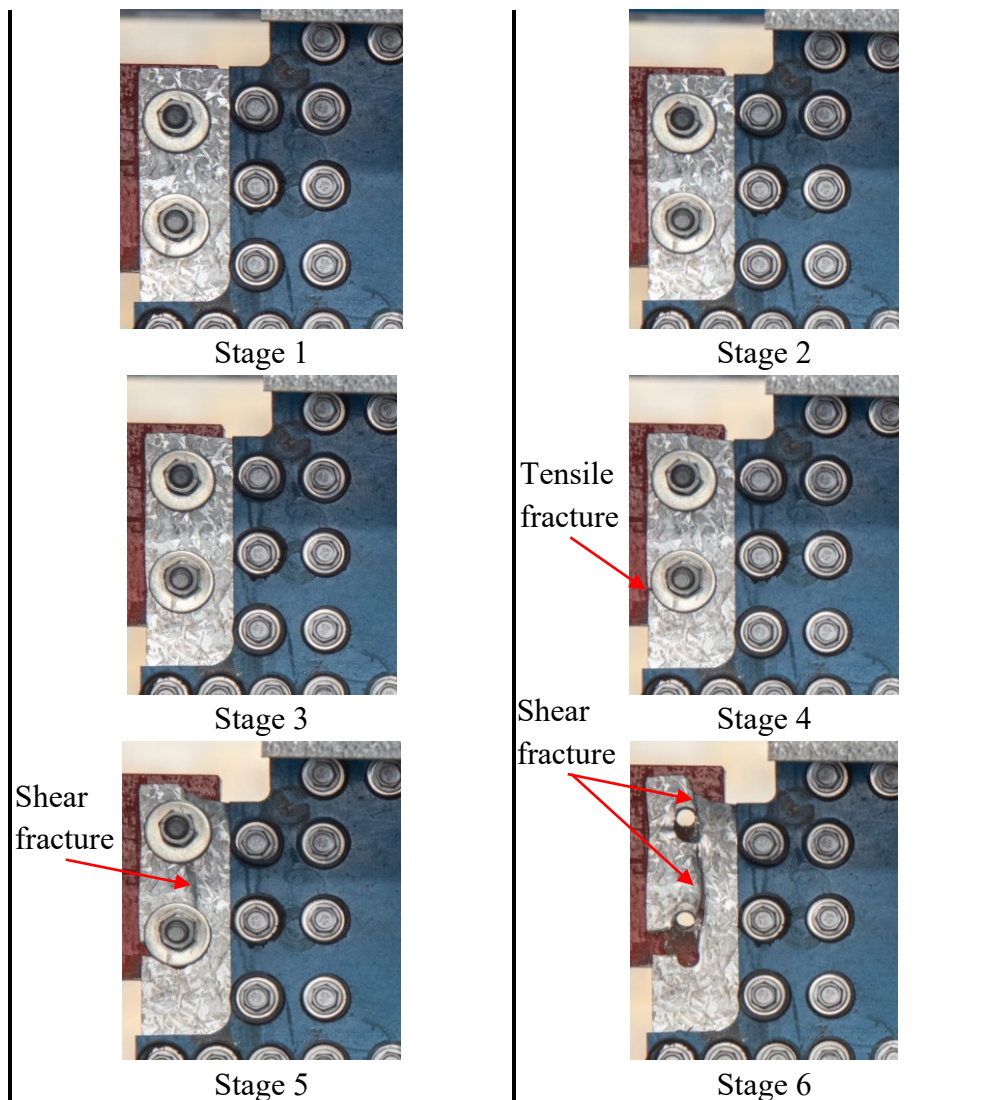
Table 4.3: Test results of the 1.2mm cold-formed steel shear bolted connections with running ratio RR=1.0

Specimen	Nominal thickness t (mm)	Nominal hole size d_h (mm)	Distance s (mm)	Edge distance e_2 (mm)	Running ratio (RR)	Test results P^T (kN)	Limit state
T1.2-12-30-1.0-20-01	1.20	14	30	20	1.0	33.63	BS
T1.2-12-30-1.0-20-02	1.20	14	30	20	1.0	34.46	BS
T1.2-12-40-1.0-20-01	1.20	14	40	20	1.0	36.57	BS
T1.2-12-40-1.0-20-02	1.20	14	40	20	1.0	37.76	BS
T1.2-12-50-1.0-20-01	1.20	14	50	20	1.0	40.11	BS
T1.2-12-50-1.0-20-02	1.20	14	50	20	1.0	42.04	BS
T1.2-12-30-1.0-25-01	1.20	14	30	25	1.0	39.01	BS
T1.2-12-30-1.0-25-02	1.20	14	30	25	1.0	41.20	BS
T1.2-12-40-1.0-25-01	1.20	14	40	25	1.0	45.07	BS
T1.2-12-40-1.0-25-02	1.20	14	40	25	1.0	44.26	BS
T1.2-12-50-1.0-25-01	1.20	14	50	25	1.0	47.45	BS
T1.2-12-50-1.0-25-02	1.20	14	50	25	1.0	47.69	BS
T1.2-12-30-1.0-30-01	1.20	14	30	30	1.0	43.87	BS
T1.2-12-30-1.0-30-02	1.20	14	30	30	1.0	45.05	BS
T1.2-12-40-1.0-30-01	1.20	14	40	30	1.0	46.98	BS
T1.2-12-40-1.0-30-02	1.20	14	40	30	1.0	47.43	BS
T1.2-12-50-1.0-30-01	1.20	14	50	30	1.0	51.50	BS
T1.2-12-50-1.0-30-02	1.20	14	50	30	1.0	50.70	BS
T1.2-08-20-1.0-15-01	1.20	09	20	15	1.0	24.83	BS
T1.2-08-20-1.0-15-02	1.20	09	20	15	1.0	25.49	BS
T1.2-08-30-1.0-15-01	1.20	09	30	15	1.0	28.47	BS
T1.2-08-30-1.0-15-02	1.20	09	30	15	1.0	28.54	BS
T1.2-08-40-1.0-15-01	1.20	09	40	15	1.0	32.57	BS
T1.2-08-40-1.0-15-02	1.20	09	40	15	1.0	32.62	BS
T1.2-08-20-1.0-20-01	1.20	09	20	20	1.0	28.79	BS
T1.2-08-20-1.0-20-02	1.20	09	20	20	1.0	29.26	BS
T1.2-08-30-1.0-20-01	1.20	09	30	20	1.0	33.18	BS
T1.2-08-30-1.0-20-02	1.20	09	30	20	1.0	32.50	BS
T1.2-08-40-1.0-20-01	1.20	09	40	20	1.0	37.33	BE
T1.2-08-40-1.0-20-02	1.20	09	40	20	1.0	37.56	BE
T1.2-08-20-1.0-25-01	1.20	09	20	25	1.0	32.28	BS
T1.2-08-20-1.0-25-02	1.20	09	20	25	1.0	32.82	BS
T1.2-08-30-1.0-25-01	1.20	09	30	25	1.0	36.84	BE
T1.2-08-30-1.0-25-02	1.20	09	30	25	1.0	37.66	BE

Note: BS – Block shear; BE – Bearing; BF – Bolt fracture



(a)

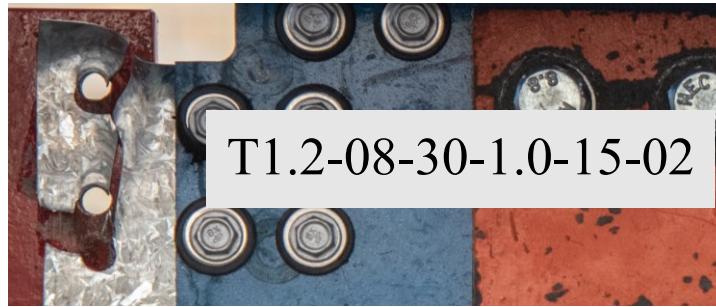


(b)

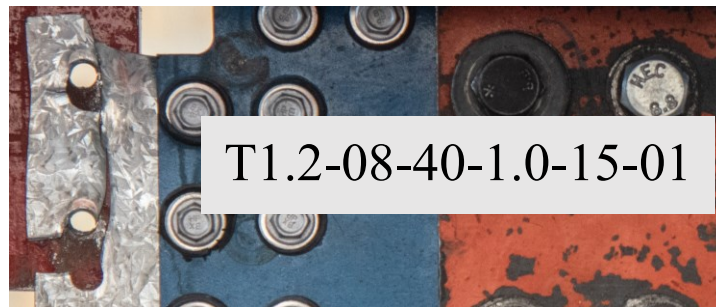
Figure 4.10: (a) Test results of 1.2mm cold-formed steel channels using M8-bolts with $e_2 = 15mm$
(b) Fracture process of the specimen T1.2-08-40-1.0-15-01



(a)



(b)



(c)

Figure 4.11: Test results of 1.2mm cold-formed steel channels using M8-bolts with $e_2 = 15mm$ (a) shear distance $s = 20mm$, (b) shear distance $s = 30mm$ and (c) shear distance $s = 40mm$

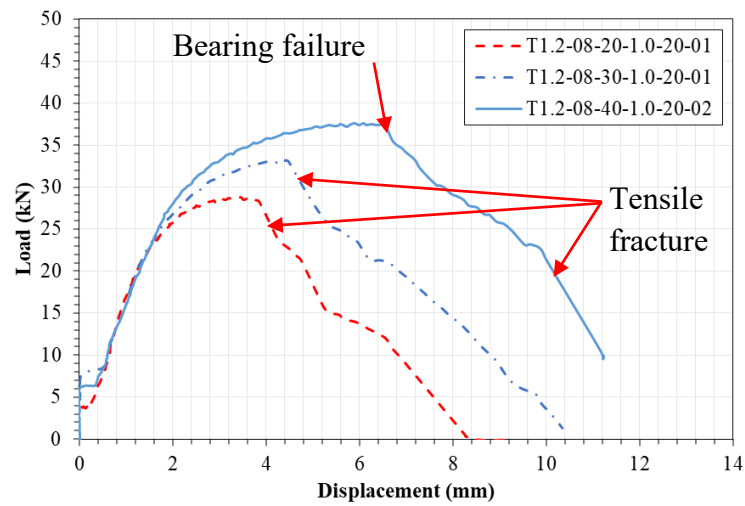
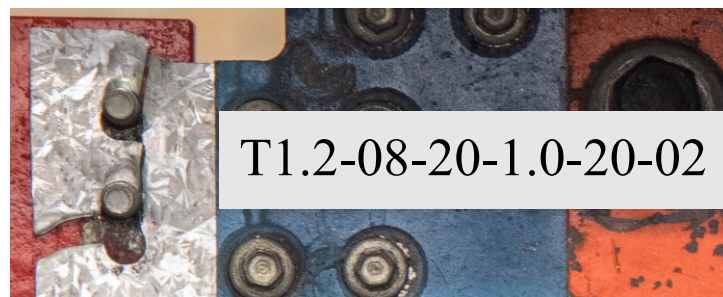


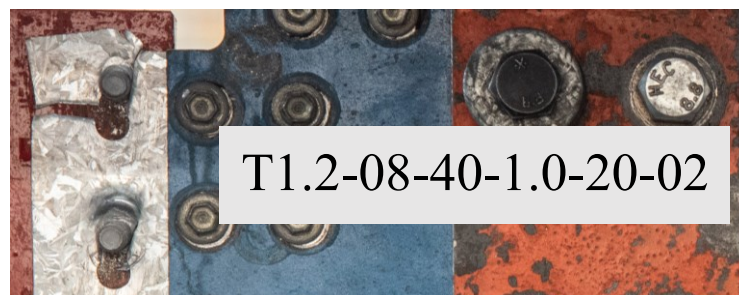
Figure 4.12: (a) Test results of 1.2mm cold-formed steel channels using M8-bolts with $e_2 = 20mm$



(a)



(b)



(c)

Figure 4.13: Test results of 1.2mm cold-formed steel channels using M8-bolts with $e_2 = 20mm$
(a) shear distance $s = 20mm$, (b) shear distance $s = 30mm$ and (c) shear distance $s = 40mm$

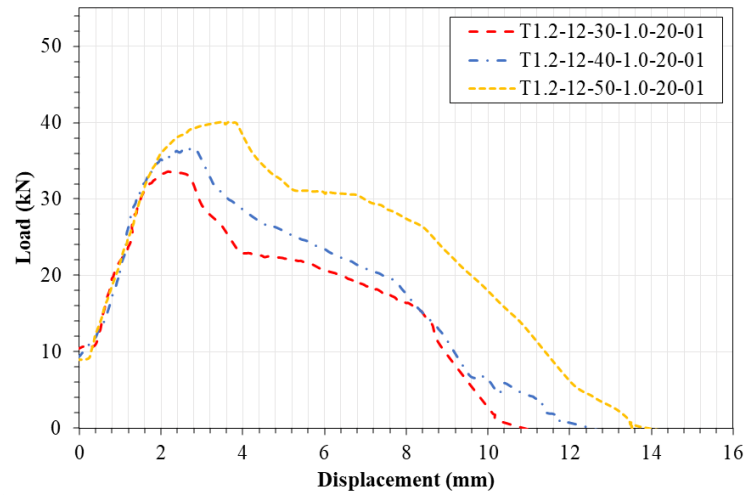
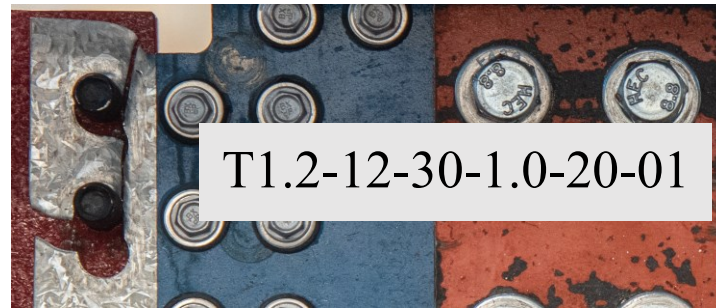
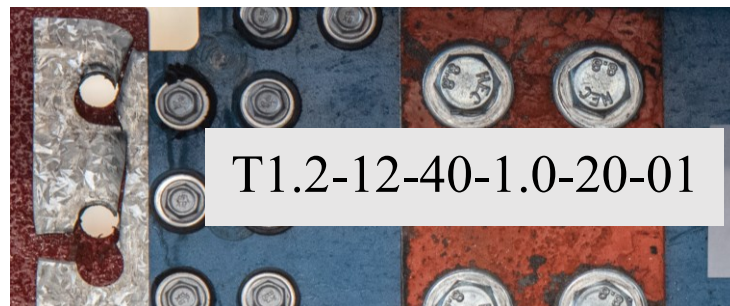


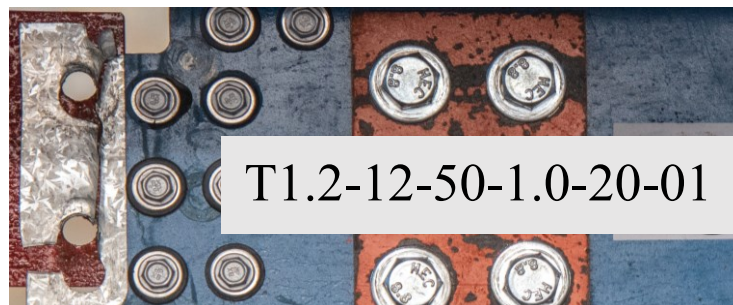
Figure 4.14: Test results of 1.2mm cold-formed steel channels using M12-bolts with $e_2 = 20mm$



(a)



(b)



(c)

Figure 4.15: Test results of 1.2mm cold-formed steel channels using M12-bolts with $e_2 = 20mm$

(a) shear distance $s = 30mm$, (b) shear distance $s = 40mm$ and (c) shear distance $s = 50mm$

4.3.1.2 Shear bolted connections tests of 1.5mm cold-formed steel channels

By following the same experimental procedure, the behaviour of the shear bolted connections was further examined to investigate the influence of thickness on the block shear strength and fracture behaviour of the connections. In this configuration, specimens were fabricated with thickness of 1.5mm and the shear distances (s) between the two bolts was the main variable. Table 4.4 provides the test data of the experiments, which further confirms the consistency of the test results. Again, the shear distance between two bolts also affected the shear strength and fracture process of the connections in both cases of different hole sizes (i.e., 14mm and 9mm). The shear strength of the shear bolted connections using M12-bolts with $e_2 = 20mm$ increased by roughly 12% corresponding to the increase of shear distance s between 30mm and 40mm while a smaller improvement with 7% was observed from $s = 40mm$ to $s = 50mm$. Similar observation occurred in the remaining cases with longer edge distances (i.e., $e_2 = 25mm$ and $30mm$), which further demonstrated the minor effect of the edge distance e_2 on the block shear behaviour of the shear beam-end bolted connections.

Likewise in the cases of the smaller M8-high strength steel bolts with $e_2 = 15mm$, similar behaviour was observed with a slightly larger improvement in the shear strength of the connections. Nevertheless, in the cases of bolted connections with longer edge distances (i.e., specimens T1.5-08-40-1.0-20-01 and T1.5-08-40-1.0-20-02), shear fracture in the bolts occurred where block shear capacity was higher than the bolt shear strength capacity, which was not of interest to investigate further in these configurations.

Figure 4.16a shows the relationship between the displacement and the block shear strength of the shear bolted connections using M12-bolts with $e_2 = 20mm$. In this particular configuration, typical block shear ruptures were observed for the three cases with various shear distances ($s = 30mm$, $40mm$ and $50mm$) even though the fracture processes were different. Similar to the first series of tests (i.e., 1.2mm CFS channels), the shear bolted connections with the shear distance $s = 50mm$ also demonstrated the later occurrence of the shear fracture between the bolts in comparison with the tensile fracture at the bottom bolt by using a relatively long shear distance (s). On the other hand, in the remaining cases (i.e., $s = 30mm$ and $40mm$), the shear fracture and tensile fracture appeared simultaneously after reaching the peak (e.g., Stage 3 in Figure 4.16a & Figure 4.16b) by using the relatively short shear distances (s) as can be seen in Stage 4 and 5 in Figure 4.16a & Figure 4.16b. These behaviours are similar to those observed in the first series of tests (i.e., 1.2mm CFS channels), which further confirms the dependency of the shear strength and fracture process on the shear distance s . Last but not least, the shear strength of shear bolted connections using 1.5mm CFS channels was higher than that using 1.2mm CFS channels. In

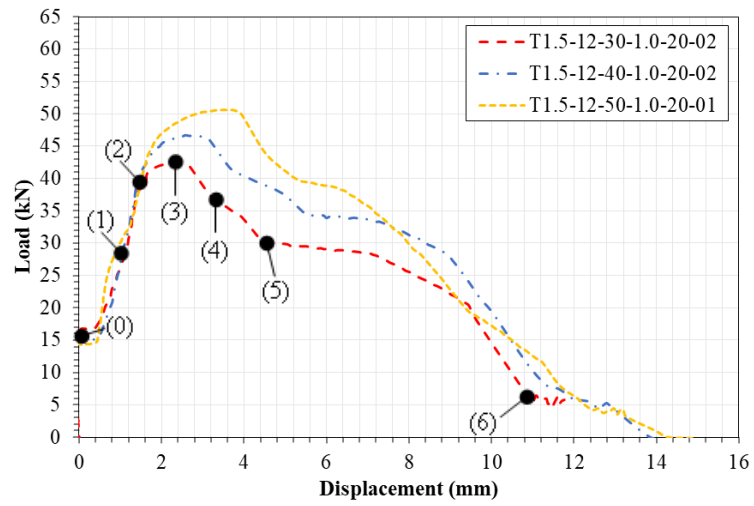
addition, the active shear resistance planes are also confirmed in Figure 4.17 for three different shear distances ($s = 30mm$, $40mm$ and $50mm$).

Similar behaviour was also observed in the shear bolted connections using M8-bolts with $e_2 = 15mm$ as shown in Figure 4.18 - Figure 4.19. Shear fracture in bolts of the specimen T1.5-08-40-1.0-20-01 is also demonstrated in Figure 4.20 after bearing failure. Details of full set of load-displacement curves are given in Appendix C.

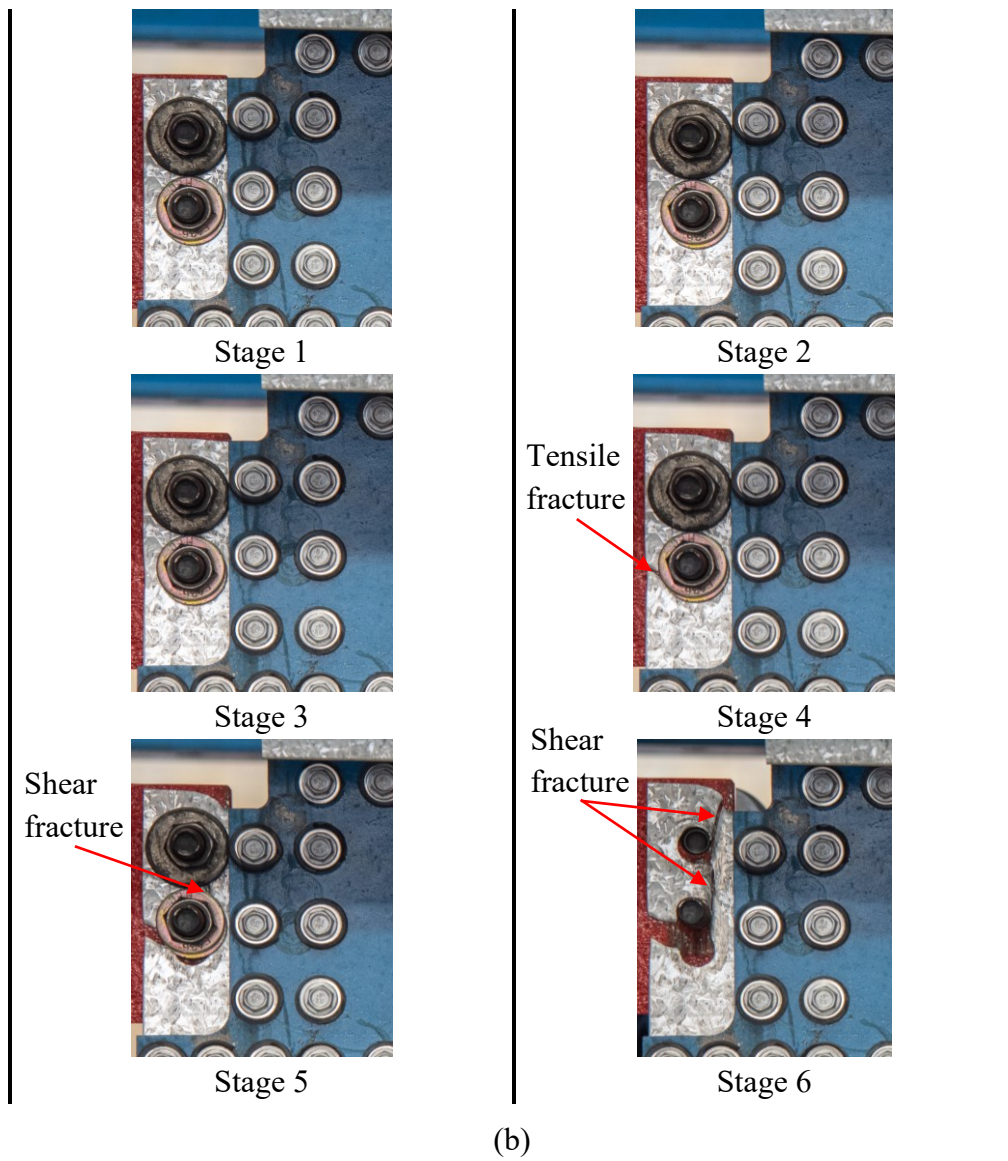
Table 4.4: Test results of the 1.5mm cold-formed steel shear bolted connections with running ratio $RR=1.0$

Specimen	Nominal thickness t (mm)	Nominal hole size d_h (mm)	Distance s (mm)	Edge distance e_2 (mm)	Running ratio (RR)	Test results P^T (kN)	Limit state
T1.5-12-30-1.0-20-01	1.50	14	30	20	1.0	41.96	BS
T1.5-12-30-1.0-20-02	1.50	14	30	20	1.0	42.34	BS
T1.5-12-40-1.0-20-01	1.50	14	40	20	1.0	47.56	BS
T1.5-12-40-1.0-20-02	1.50	14	40	20	1.0	46.67	BS
T1.5-12-50-1.0-20-01	1.50	14	50	20	1.0	50.63	BS
T1.5-12-50-1.0-20-02	1.50	14	50	20	1.0	50.51	BS
T1.5-12-30-1.0-25-01	1.50	14	30	25	1.0	44.78	BS
T1.5-12-30-1.0-25-02	1.50	14	30	25	1.0	45.48	BS
T1.5-12-40-1.0-25-01	1.50	14	40	25	1.0	49.36	BS
T1.5-12-40-1.0-25-02	1.50	14	40	25	1.0	50.06	BS
T1.5-12-50-1.0-25-01	1.50	14	50	25	1.0	54.10	BS
T1.5-12-50-1.0-25-02	1.50	14	50	25	1.0	54.19	BS
T1.5-12-30-1.0-30-01	1.50	14	30	30	1.0	49.51	BS
T1.5-12-30-1.0-30-02	1.50	14	30	30	1.0	50.39	BS
T1.5-12-40-1.0-30-01	1.50	14	40	30	1.0	54.34	BS
T1.5-12-40-1.0-30-02	1.50	14	40	30	1.0	53.84	BS
T1.5-12-50-1.0-30-01	1.50	14	50	30	1.0	57.17	BS
T1.5-12-50-1.0-30-02	1.50	14	50	30	1.0	58.23	BS
T1.5-08-20-1.0-15-01	1.50	09	20	15	1.0	28.86	BS
T1.5-08-20-1.0-15-02	1.50	09	20	15	1.0	29.64	BS
T1.5-08-30-1.0-15-01	1.50	09	30	15	1.0	34.79	BS
T1.5-08-30-1.0-15-02	1.50	09	30	15	1.0	34.18	BS
T1.5-08-40-1.0-15-01	1.50	09	40	15	1.0	37.22	BS
T1.5-08-40-1.0-15-02	1.50	09	40	15	1.0	38.52	BS
T1.5-08-20-1.0-20-01	1.50	09	20	20	1.0	34.48	BS
T1.5-08-20-1.0-20-02	1.50	09	20	20	1.0	35.16	BS
T1.5-08-30-1.0-20-01	1.50	09	30	20	1.0	39.88	BS
T1.5-08-30-1.0-20-02	1.50	09	30	20	1.0	39.75	BS
T1.5-08-40-1.0-20-01	1.50	09	40	20	1.0	42.43	BF
T1.5-08-40-1.0-20-02	1.50	09	40	20	1.0	41.97	BF
T1.5-08-20-1.0-25-01	1.50	09	20	25	1.0	37.54	BS
T1.5-08-20-1.0-25-02	1.50	09	20	25	1.0	37.49	BS

Note: BS – Block shear; BE – Bearing; BF – Bolt fracture

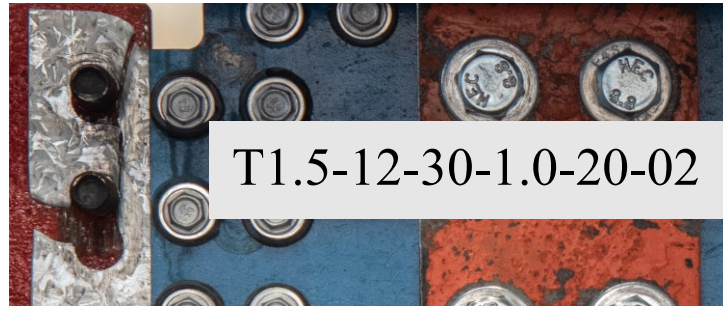


(a)

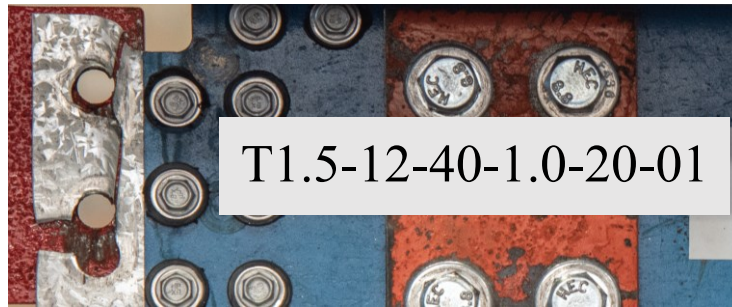


(b)

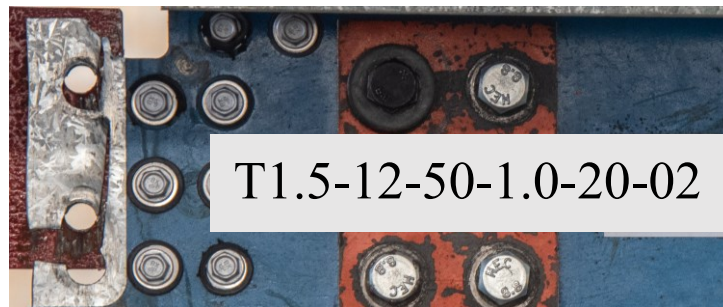
Figure 4.16: (a) Test results of 1.5mm cold-formed steel channels using M12-bolts with $e_2 = 20mm$ (b) Fracture process of the specimen T1.5-12-30-1.0-20-02



(a)



(b)



(c)

Figure 4.17: Test results of 1.5mm cold-formed steel channels using M12-bolts with $e_2 = 20mm$

(a) shear distance $s = 30mm$, (b) shear distance $s = 40mm$ and (c) shear distance $s = 50mm$

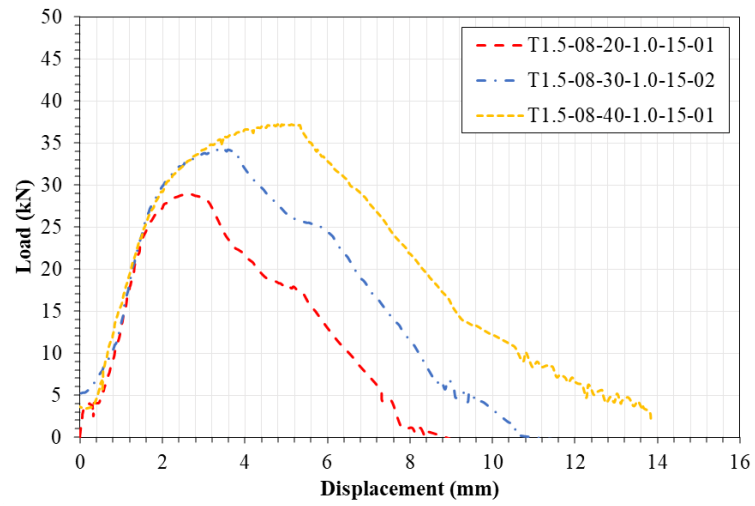
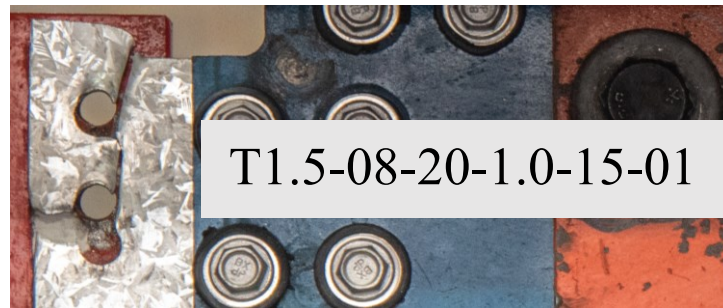
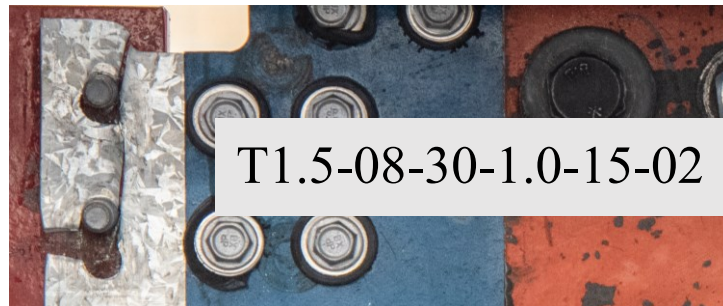


Figure 4.18: Test results of 1.5mm cold-formed steel channels using M8-bolts with $e_2 = 15mm$



(a)

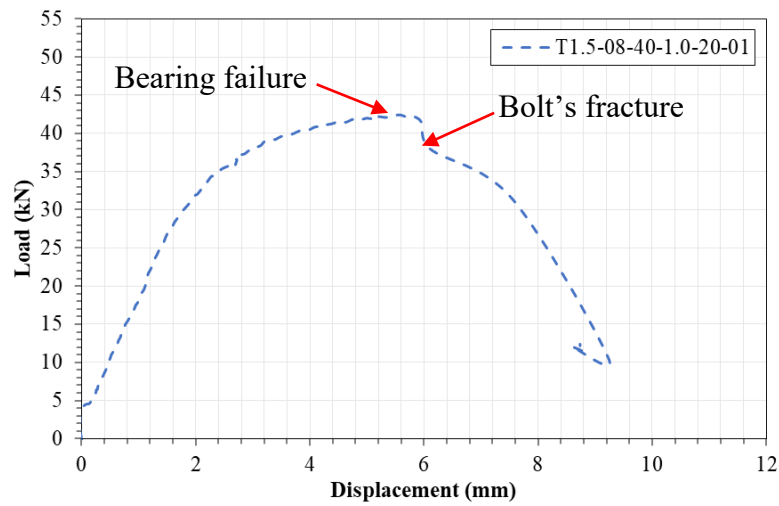


(b)

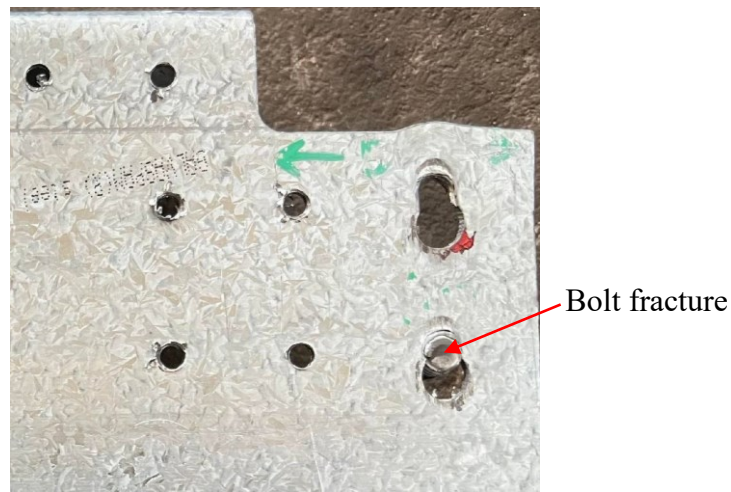


(c)

Figure 4.19: Test results of 1.5mm cold-formed steel channels using M8-bolts with $e_2 = 15mm$ (a) shear distance $s = 20mm$, (b) shear distance $s = 30mm$ and (c) shear distance $s = 40mm$



(a)



(b)

Figure 4.20: (a) Load-displacement curve and (b) Shear fracture in bolts of the specimen T1.5-08-40-1.0-20-01 captured on the other side

4.3.2. Running ratios $RR=2.0$ and $RR=3.0$

Similar experimental procedures were employed for the shear beam-end bolted connections using different running ratios (RR) to further investigate the influence of the rotations on the block shear strength and fracture behaviour of the connections. In particular, specimens were fabricated with the same configurations as conducted in the previous sections while the ratio of speed of Actuator A over the speed of Actuator B varied between 1.0, 2.0 or 3.0. The test results of the shear beam-end bolted connections using different running ratios is given in Table 4.5. It is noticeable that the shear strength of the connections using $RR=2.0$ roughly equalled to that using $RR=1.0$. However, in terms of $RR=3.0$ (i.e., specimens T1.2-12-40-3.0-25-01 and T1.2-12-40-3.0-25-02), the connections not only provided a different block shear limit state (i.e., shear fracture occurred along different paths) but also demonstrated a large deformation (Figure 4.21) in

comparison with the others (Figure 4.22) which was not appropriate for practical design and therefore it was not of interest to investigate further the running ratio of 3.0.

In addition, the relationship between load and displacement of the shear beam-end bolted connections using different running ratios is illustrated in Figure 4.23 - Figure 4.24. Clearly, the difference in the shear capacities of the specimen T1.2-12-40-3.0-25-01 (RR=3.0) and the specimen T1.2-12-40-1.0-25-01 (RR=1.0) is also confirmed in Figure 4.23 because of the different block shear limit state and the large deformation and rotation of the connections.

On the other hand, the load-displacement curves of the specimens T1.5-08-30-2.0-15-01 (RR=2.0) and T1.5-08-30-1.0-15-01 (RR=1.0) were nearly indistinguishable. Similar behaviour was observed in the remaining cases of the bolted connections which illustrated the minor effect of the rotation on the block shear capacity and fracture behaviour of the shear beam-end bolted connections. The difference between the bolt size and hole size in the bolted connections was relatively high causing a high initial slip and rotation of the connections during the loading. Moreover, the shear capacity of the bolted connections only depended on the fracture paths which did not change in the cases of bolted connections using the higher running ratio (RR=2.0). By comparison, the rotation significantly affected the shear strength of the screw connections (Huynh et al. [46]) and shear strength of the PAF connections (Truong et al. [47]) where bearing failure dominated the failure mode in the same test rig. Details of full set of load-displacement curves are given in Appendix C.

Table 4.5: Test results of the cold-formed steel shear bolted connections with running ratios RR=2.0 and RR=3.0

Specimen	Nominal thickness t (mm)	Nominal hole size d_h (mm)	Distance s (mm)	Edge distance e_2 (mm)	Running ratio (RR)	Test results P^T (kN)	Limit state
T1.2-12-30-2.0-30-01	1.20	14	30	30	2.0	41.75	BS
T1.2-12-30-2.0-30-02	1.20	14	30	30	2.0	41.30	BS
T1.2-12-40-3.0-25-01	1.20	14	40	25	3.0	39.39	BS*
T1.2-12-40-3.0-25-02	1.20	14	40	25	3.0	41.01	BS*
T1.2-12-50-2.0-25-01	1.20	14	50	25	2.0	45.77	BS
T1.2-12-50-2.0-25-02	1.20	14	50	25	2.0	45.27	BS
T1.2-08-30-2.0-20-01	1.20	09	30	20	2.0	34.57	BS
T1.2-08-30-2.0-20-02	1.20	09	30	20	2.0	33.67	BS
T1.5-12-40-2.0-25-01	1.50	14	40	25	2.0	51.73	BS
T1.5-12-40-2.0-25-02	1.50	14	40	25	2.0	48.35	BS
T1.5-08-30-2.0-15-01	1.50	09	30	15	2.0	34.15	BS
T1.5-08-30-2.0-15-02	1.50	09	30	15	2.0	34.10	BS

Note: BS – Block shear; BS* – Different block shear limit state

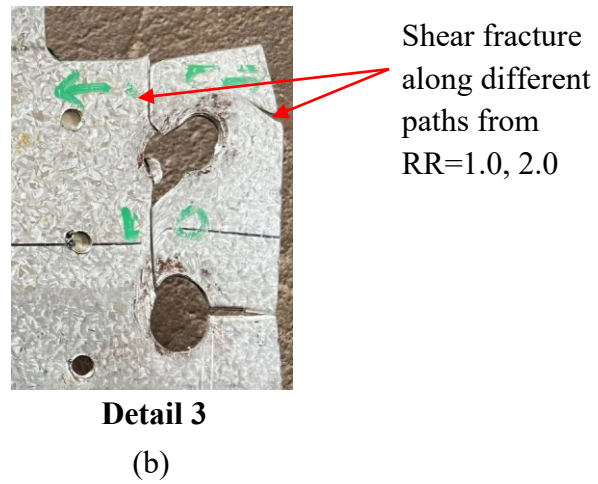
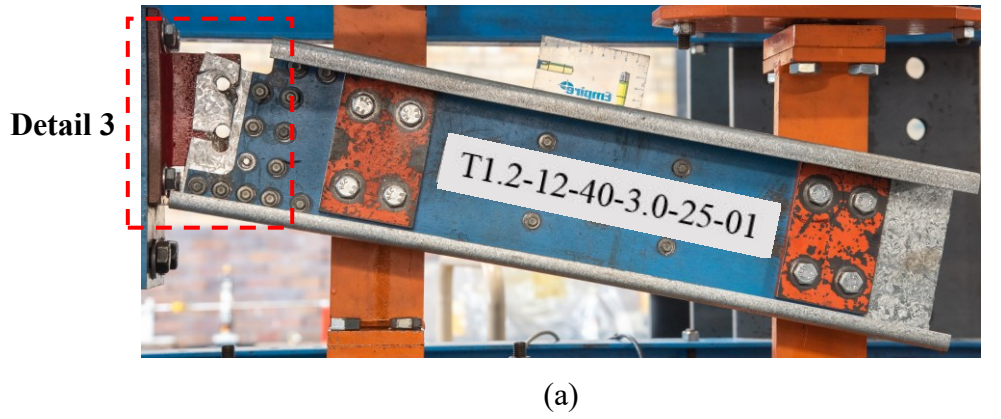


Figure 4.21: Deformation and fracture behaviour of the specimen T1.2-12-40-3.0-25-01 with RR=3.0 captured on the other side

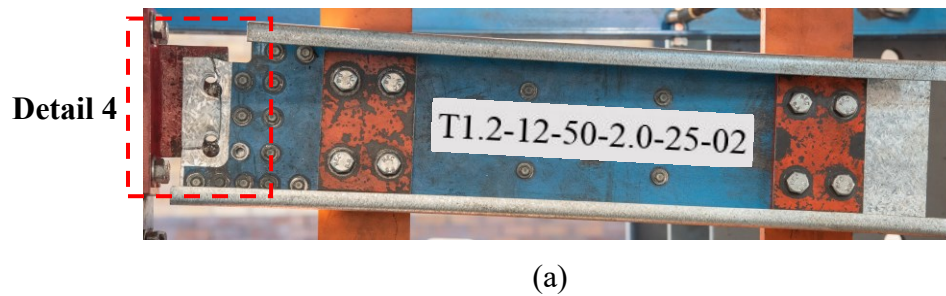


Figure 4.22: Deformation and fracture behaviour of the specimen T1.2-12-50-2.0-25-02 with RR=2.0

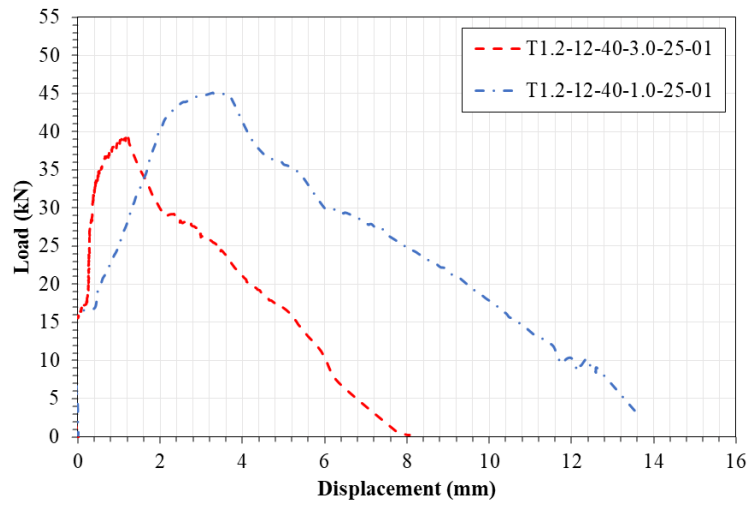


Figure 4.23: Comparison of the test results of the specimen T1.2-12-40-RR-25 using different running ratios

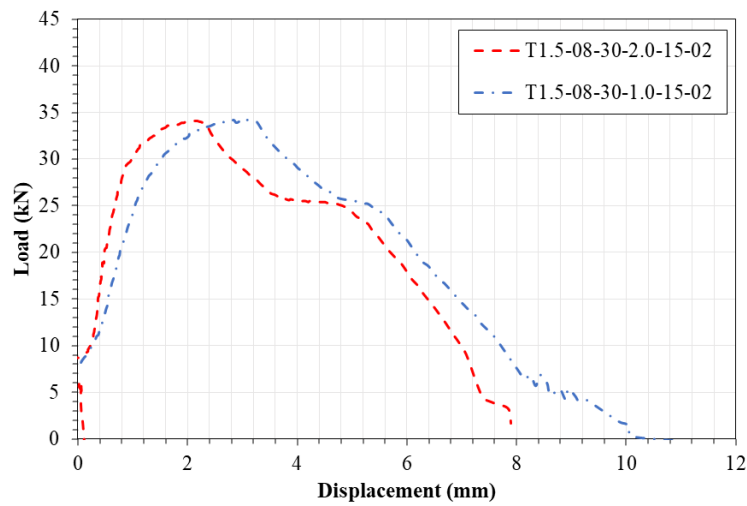


Figure 4.24: Comparison of the test results of the specimen T1.5-08-30-RR-15 using different running ratios

4.4. DESIGN STANDARD AND SPECIFICATION COMPARISONS

4.4.1. Current design equations in standards/specifications

In the current cold-formed steel design specification from the USA (AISI S100:16 (R2020) w/S3-22 – Section J6.1 [1]), the nominal strength of the bolted connections under pure shear rupture (R_{nv}) is calculated as follows

$$R_{nv} = 0.6 f_u A_{nv} \quad (4.1)$$

where f_u is the tensile strength of the connected parts; and A_{nv} is the net area subject to shear (parallel to force). For a connection where each individual fastener pulls through the material towards the limiting edge individually

$$A_{nv} = 2nte_{net} \quad (4.2)$$

in which n denotes the number of fasteners on critical cross-section; t is the base steel thickness of section; and e_{net} is the clear distance between the end of the material and the edge of the fastener hole or weld. In the case of coped beam-end connection

$$A_{nv} = (h_{wc} - n_b d_h) t \quad (4.3)$$

where h_{wc} is the coped flat web depth; n_b denotes the number of fasteners along failure path; d_h is the diameter of the hole; and t is the thickness of coped web.

Under pure tension rupture, the nominal strength of the bolted connections (R_{nt}) shall be calculated from AISI S100:16 (R2020) w/S3-22 – Section J6.2 [1] as follows

$$R_{nt} = U_{sl} f_u A_{nt} \quad (4.4)$$

in which $U_{sl} = (0.9 + 0.1 d_f / s_f)$ is the shear lag factor; the variable d_f denotes the bolt diameter; and s_f denotes the bolt spacing in the tensile resistance plane and A_{nt} is the net area subject to tension (perpendicular to force).

In addition, the block shear strength of conventional rectangular block shear bolted connections is specified. In particular, the nominal block shear rupture strength (R_n) in AISI S100:16 (R2020) w/S3-22 – Section J6.3 [1] is determined as follows

$$R_n = 0.6 f_y A_{av} + U_{bs} U_{sl} f_u A_{nt} \quad (4.5)$$

Where $A_{av} = (A_{gv} + A_{nv}) / 2$ is the active shear area lying in the middle of the net and gross shear areas; f_y is the yield stress of the connected parts; U_{bs} is a non-uniform block shear factor (0.5 for coped beam shear conditions with more than one vertical row of connectors, 1.0 for all other cases); and gross area subject to shear is denoted by A_{gv} .

In the Australian and New Zealand Standard (AS/NZS 4600:2018 [2]), the design Equation (4.6) for block shear failure using an active shear area concept for the shear zones is the same as that in AISI S100:16 (R2020) w/S3-22 – Section J6.3 [1]. The Australian and New Zealand Standard was first to include this equation in 2018 while the AISI Specification followed in 2022 with Supplement S3. Nevertheless, the proposal is only employed based on simple connections without any test results for the shear beam-end bolted connections

$$R_n = 0.6 f_y A_{av} + f_u A_{nt} (0.9 + 0.1 d_f / s_f) \quad (4.6)$$

in which A_{av} is the active shear area lying in the middle of the net and gross shear areas.

Likewise, in Eurocode 3 Part 1.8 [15], the design equation for block shear failure without considering the presence of the active shear area is given by Equation (4.7)

$$R_n = (1/\sqrt{3})f_y A_{nv} + f_u A_{nt} \quad (4.7)$$

4.4.2. Effect of the shear distance on the behaviour of the fracture process in the shear zones

Based on the experimental results in this chapter, the shear distance (s) between the bolts not only affects the shear strength of the connections but also changes the fracture process behaviour in the shear zones. In the case of the relatively short shear distance (s), the shear fracture and tensile fracture appear simultaneously after reaching the peak whereas longer shear distances (s) demonstrate the later occurrence of the shear fracture between the bolts in comparison with the tensile fracture at the bottom bolt. This behaviour is demonstrated in Figure 4.25. In terms of the relatively short shear distance (s), the ultimate tensile stress and ultimate shear stress localize respectively around the tension zone and shear zone between the two bolts. Therefore, after the tensile fracture occurs at the bottom bolt in Region 3, the shear fracture between the two bolts also appears simultaneously in Region 2. This phenomenon occurs when the shear yielding strength ($R_y^S = 0.6f_y A_{av}$) in Region 2 between the two bolts is less than the tensile fracture strength ($R^T = U_{sl}f_u A_{nt}$) in Region 3 (i.e., $R_y^{S(2)} < R^{T(3)}$). However, the reverse behaviour is apparent in the connections using the relatively long shear distances (s). For instance, the ultimate tensile stress localizes around the tension zone while the shear stress in the shear zone between the two bolts does not achieve its ultimate value. Consequently, after the tensile fracture occurs at the bottom bolt in Region 3, shear yielding starts to occur in the shear zone between the two bolts in Region 2. This phenomenon occurs when the shear yielding strength ($R_y^S = 0.6f_y A_{av}$) in Region 2 between the two bolts is larger than or equal to the tensile fracture strength ($R^T = U_{sl}f_u A_{nt}$) in Region 3 (i.e., $R_y^{S(2)} \geq R^{T(3)}$). These behaviours are similar to that observed in the experimental results in the previous sections and the proposal in Kim et al. [3] and Kim et al. [57].

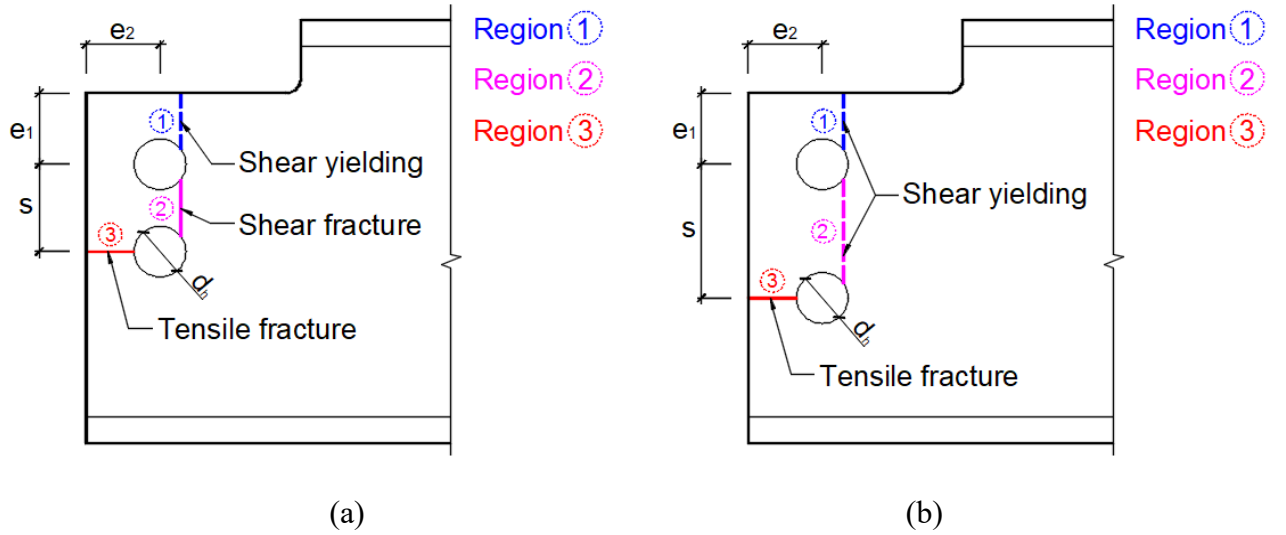


Figure 4.25: Two cases of the block shear fracture process behaviour: (a) Case 1 with the shear distance s relatively short and (b) Case 2 with the shear distance s relatively long

4.4.3. Comparison of the experimental results with the cold-formed steel design standard in Australia and New Zealand Standard (AS/NZS 4600:2018)

Table 4.6 shows the comparison of the test results with the cold-formed steel design standard in the Australia and New Zealand Standard (AS/NZS 4600:2018 [2]) to further demonstrate the accuracy of the cold-formed steel design standard AS/NZS 4600:2018 [2] for the block shear capacity of the shear beam-end bolted connections under the combined effects of shear and rotation. The ratio of the average test results divided by the predicted values ranges from 1.02 to 1.21. It is worth pointing out that AS/NZS 4600:2018 [2] provides a good agreement with the experimental results via the average ratios $P^T/R_n^{AS/NZS}$ and the corresponding coefficient of variation (i.e., Mean = 1.11 and C.o.V = 4.32%) although the cold-formed steel design standard AS/NZS 4600:2018 does not consider the effect of the shear distance (s) on the fracture process behaviour of the connections as observed from the experiments because the shear fracture in Region 2 in Figure 4.25 only occurs with the shear distance (s) relatively short. Furthermore, the difference between yield stress (f_y) and tensile strength (f_u) is relatively small by using high strength G450 and G500 steels with significantly lower ductility. As a result, the difference between the shear yielding strength ($R_y^S = 0.6f_yA_{av}$) and the shear fracture strength ($R^T = U_{sl}f_uA_m$) is not significant.

Nevertheless, the AS/NZS 4600:2018 [2] predictions are more conservative in obtaining the nominal block shear strength against the experimental results without considering the fracture process behaviour in the shear zones in the cases of the bolted connections where the edge distance (e_2) is relatively long while the shear distance (s) is relatively short (i.e., T1.2-12-30-

1.0-25, T1.2-12-30-1.0-30, T1.2-08-20-1.0-20, T1.5-12-30-1.0-20, T1.5-12-30-1.0-25, T1.5-12-30-1.0-30, etc.). The comparison of the test results with the cold-formed steel design standard in the Australia and New Zealand Standard (AS/NZS 4600:2018 [2]) is further demonstrated in Figure 4.26 - Figure 4.29.

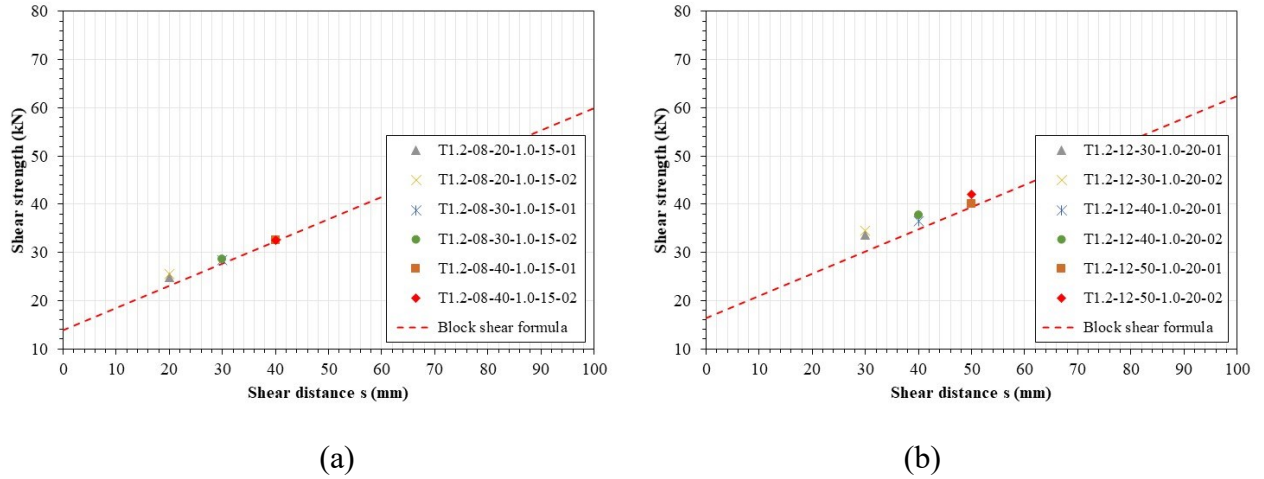


Figure 4.26: Comparison of the experimental results with the cold-formed steel design standard AS/NZS 4600:2018 [2] of the 1.2mm cold-formed steel bolted connections (a) using M8-bolts with $e_2 = 15mm$ and (b) using M12-bolts with $e_2 = 20mm$

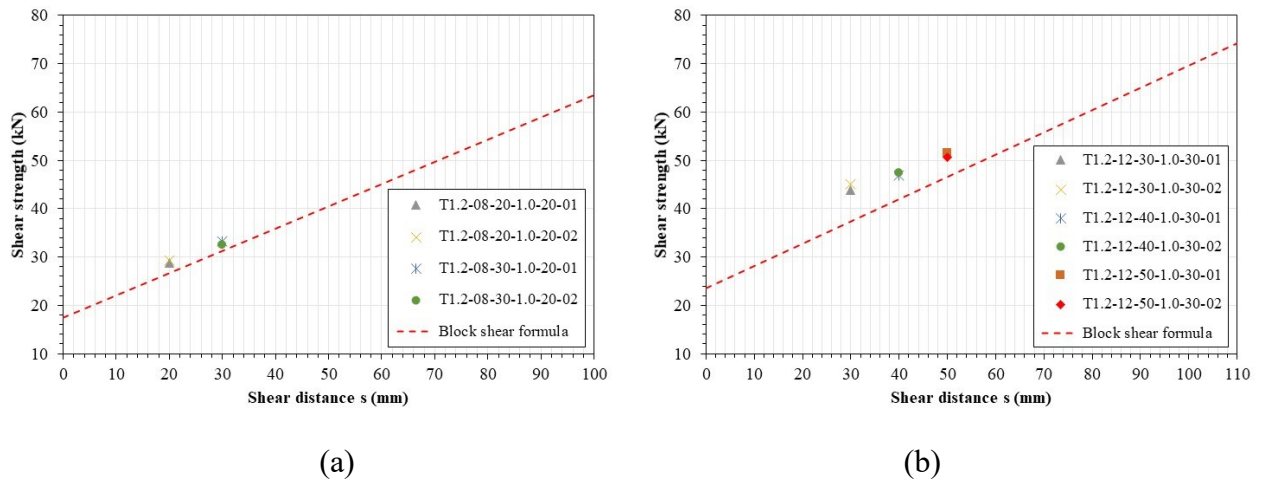
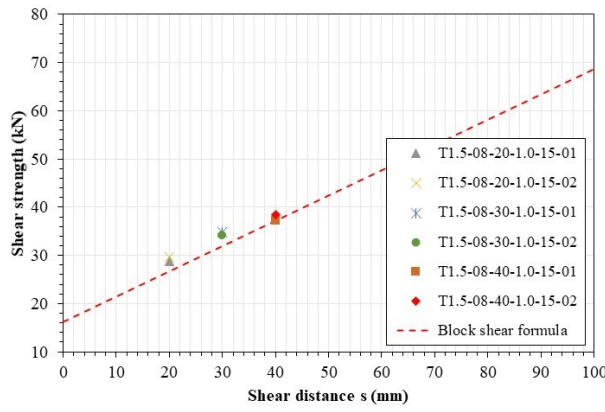
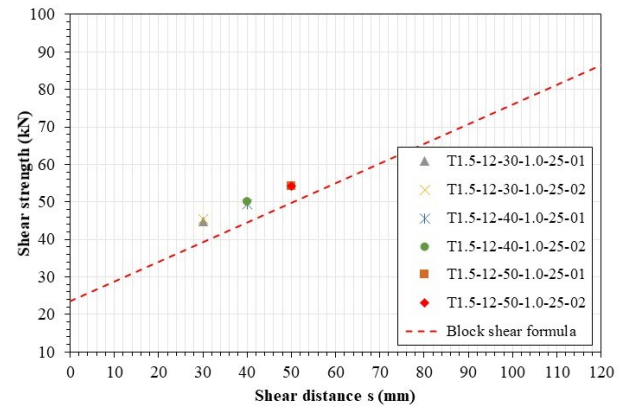


Figure 4.27: Comparison of the experimental results with the cold-formed steel design standard AS/NZS 4600:2018 [2] of the 1.2mm cold-formed steel bolted connections (a) using M8-bolts with $e_2 = 20mm$ and (b) using M12-bolts with $e_2 = 30mm$

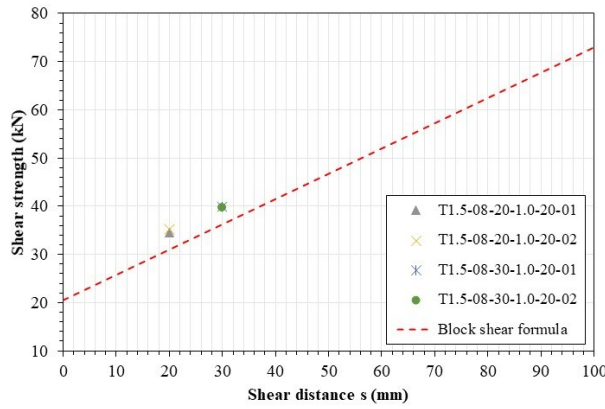


(a)

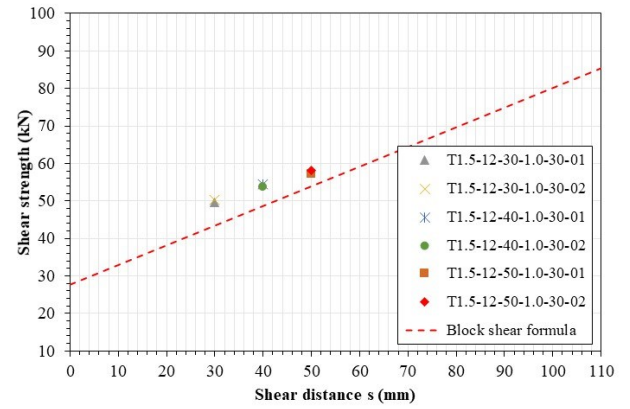


(b)

Figure 4.28: Comparison of the experimental results with the cold-formed steel design standard AS/NZS 4600:2018 [2] of the 1.5mm cold-formed steel bolted connections (a) using M8-bolts with $e_2 = 15mm$ and (b) using M12-bolts with $e_2 = 25mm$



(a)



(b)

Figure 4.29: Comparison of the experimental results with the cold-formed steel design standard AS/NZS 4600:2018 [2] of the 1.2mm cold-formed steel bolted connections (a) using M8-bolts with $e_2 = 20mm$ and (b) using M12-bolts with $e_2 = 30mm$

Table 4.6: Comparison of the experimental results with the cold-formed steel design standard in Australia and New Zealand Standard (AS/NZS 4600:2018 [2]) for the cold-formed steel bolted connections

Specimen	Nominal thickness t (mm)	Shear distance s (mm)	Edge distance e_2 (mm)	Average test results P^T (kN)	AS/NZS 4600:2018 R_n (kN)	$\frac{P^T}{R_n^{AS/NZS}}$
T1.2-12-30-1.0-20	1.20	30	20	34.05	29.92	1.14
T1.2-12-40-1.0-20	1.20	40	20	37.17	34.47	1.08
T1.2-12-50-1.0-20	1.20	50	20	41.08	39.02	1.05
T1.2-12-30-1.0-25	1.20	30	25	40.11	33.47	1.20
T1.2-12-40-1.0-25	1.20	40	25	44.67	38.02	1.17
T1.2-12-50-1.0-25	1.20	50	25	47.57	42.57	1.12
T1.2-12-30-1.0-30	1.20	30	30	44.46	36.99	1.20
T1.2-12-40-1.0-30	1.20	40	30	47.21	41.54	1.14
T1.2-12-50-1.0-30	1.20	50	30	51.10	46.10	1.11
T1.2-12-30-2.0-30	1.20	30	30	41.53	36.99	1.12
T1.2-12-50-2.0-25	1.20	50	25	45.52	42.57	1.07
T1.2-08-20-1.0-15	1.20	20	15	25.16	22.88	1.10
T1.2-08-30-1.0-15	1.20	30	15	28.51	27.43	1.04
T1.2-08-40-1.0-15	1.20	40	15	32.60	31.99	1.02
T1.2-08-20-1.0-20	1.20	20	20	29.03	26.41	1.10
T1.2-08-30-1.0-20	1.20	30	20	32.84	30.97	1.06
T1.2-08-20-1.0-25	1.20	20	25	32.55	29.93	1.09
T1.2-08-30-2.0-20	1.20	30	20	34.12	30.97	1.10
T1.5-12-30-1.0-20	1.50	30	20	42.15	34.82	1.21
T1.5-12-40-1.0-20	1.50	40	20	47.12	40.05	1.18
T1.5-12-50-1.0-20	1.50	50	20	50.57	45.28	1.12
T1.5-12-30-1.0-25	1.50	30	25	45.13	39.06	1.16
T1.5-12-40-1.0-25	1.50	40	25	49.71	44.29	1.12
T1.5-12-50-1.0-25	1.50	50	25	54.15	49.52	1.09
T1.5-12-30-1.0-30	1.50	30	30	49.95	43.27	1.15
T1.5-12-40-1.0-30	1.50	40	30	54.09	48.50	1.12
T1.5-12-50-1.0-30	1.50	50	30	57.70	53.73	1.07
T1.5-12-40-2.0-25	1.50	40	25	50.04	44.29	1.13
T1.5-08-20-1.0-15	1.50	20	15	29.25	26.65	1.10
T1.5-08-30-1.0-15	1.50	30	15	34.49	31.88	1.08
T1.5-08-40-1.0-15	1.50	40	15	37.87	37.11	1.02
T1.5-08-20-1.0-20	1.50	20	20	34.82	30.87	1.13
T1.5-08-30-1.0-20	1.50	30	20	39.82	36.10	1.10
T1.5-08-20-1.0-25	1.50	20	25	37.52	35.06	1.07
T1.5-08-30-2.0-15	1.50	30	15	34.13	31.88	1.07
					Mean	1.11
					C.o.V (%)	4.32

4.4.4. Reliability analysis

In this section, the test results of the shear beam-end bolted connections are used for the calibration procedure of the reliability indices in the cold-formed steel standards and specifications from AS/NZS 4600:2018 [2] and AISI S100:2016 (R2020) w/S3-22 [1] which was discussed previously in Chapter 3. For AISI S100:2016 (R2020) w/S3-22 [1], only the LRFD method using the resistance factor ϕ will be discussed since it is comparable to the limit state methodology in AS/NZS 4600:2018 [2].

$$\phi = \left(\frac{1.2D/L + 1.5}{1.05D/L + 1} \right) (P_m M_m F_m) \exp \left(-\beta \sqrt{C_P V_P^2 + V_M^2 + V_F^2 + V_Q^2} \right) \quad (4.8)$$

In the above equation, the value of P_m is the mean value of the test-to-prediction ratio and the corresponding coefficient of variation (V_p) which were derived from the tests. Therefore, $P_m = 1.11$ and $V_p = 0.0432$ as can be seen in Table 4.6. Remaining variables were calculated in detail in Chapter 3.

Consequently, the relationship between the reliability index (β) and the ratio of dead load to live load (D/L) for different values of the resistance factor (ϕ) is demonstrated in Figure 4.30. It can be seen that a lower value of the resistance factor (ϕ) provides a higher index of the reliability (β). However, a higher value of the reliability index (β) can be obtained when using a higher value of the ratio of (D/L).

In addition, for the purposes of checking the reliability based on LRFD criteria, it has been assumed that $D/L = 0.2$. This ratio is typical for light weight CFS structures. According to AISI S100:2016 (R2020) w/S3-22 – Section K2.1 [1], a target reliability index for CFS connections for LRFD equal to 3.5 so that failure of a structure is less likely to be initiated in the connections. Table 4.7 illustrates the resistance factors (ϕ) for rupture of the bolted connections when a ratio of $D/L = 0.2$ and $\beta = 3.5$ are assumed against the cold-formed steel standards and specifications from AS/NZS 4600:2018 [2] and AISI S100:2016 (R2020) w/S3-22 [1]. This resistance factor value (i.e., $\phi = 0.90$) is significantly higher than that employed by AS/NZS 4600:2018 [2] and AISI S100:2016 (R2020) w/S3-22 [1] (i.e., $\phi = 0.80$ and $\phi = 0.75$, respectively) because the test results in this study are consistent (i.e., C.o.V = 0.0432 as shown in Table 4.6) based on the good quality control of the materials involved in the tests. Therefore, a good testing control may increase the average test-to-prediction ratio (P_m) and reduce the corresponding coefficient of variation (V_p) resulting in an even higher resistance factor (ϕ).

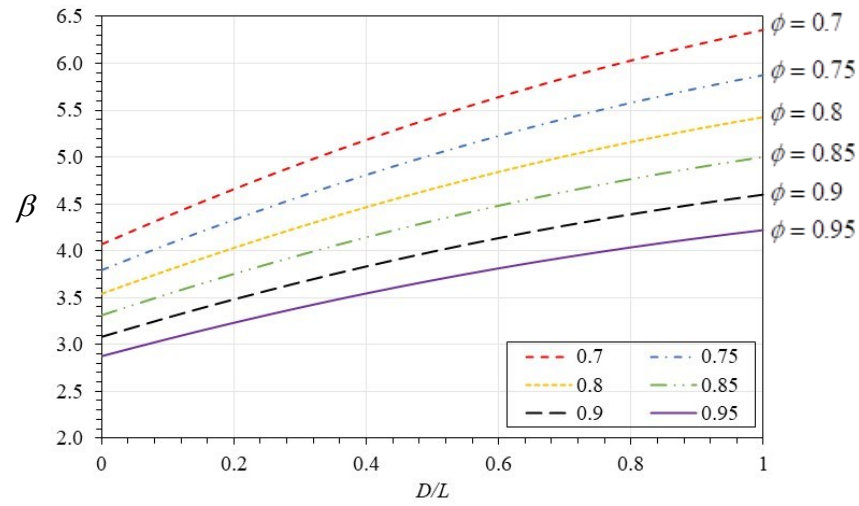


Figure 4.30: Reliability index β with ratio of dead load to live load D/L for different values of the resistance factor ϕ

Table 4.7: Resistance factor ϕ for $D/L = 0.2$ and $\beta = 3.5$

Information		AS/NZS 4600:2018 [2]	AISI S100:2016 (R2020) w/S3-22 [1]
Mean	$P_m = 1.11$		
C.o.V	$V_P = 0.0432$	$\phi = 0.90$	$\phi = 0.90$
Number of samples	$n_T = 35$	$(0.80)^*$	$(0.75)^*$

Note: * Values obtained from the Standards and Specifications

4.5. SUMMARY

A total of 78 tests were conducted in this chapter to investigate the complex behaviour of the shear beam-end bolted connections failing in the block shear mode under the combined effects of shear-rotation. Based on the research, the rotation of the channels provides a minor effect on the block shear capacity and fracture behaviour of the shear beam-end bolted connections because of the high difference between the bolt size and hole size causing a high initial slip and rotation of the connections during the loading. On the one hand, the shear capacity of the bolted connections only depends on the fracture paths which does not change in the cases of bolted connections using running ratio (RR=2.0). Nevertheless, in terms of higher running ratio (RR=3.0), the connections not only illustrate a different block shear limit state (i.e., shear fracture occurs along different paths) but also demonstrate a large deformation, which is not appropriate for practical design.

Furthermore, the shear distance (s) between the bolts not only affects the shear strength of the connections but also changes the fracture process behaviour in the shear zones observed from the experimental results. Particularly, the shear fracture and tensile fracture of the connections

appears simultaneously after reaching the peak in the case of the relatively short shear distance (s) because the ultimate tensile stress and ultimate shear stress localize respectively around the tension zone in Region 3 and shear zone between the two bolts in Region 2. This phenomenon occurs when the shear yielding strength ($R_y^S = 0.6f_y A_{av}$) in Region 2 between the two bolts is less than the tensile fracture strength ($R^T = U_{sl}f_u A_{nt}$) in Region 3 (i.e., $R_y^{S(2)} < R^{T(3)}$). However, longer shear distances (s) demonstrate the later occurrence of the shear fracture between the bolts in comparison with the tensile fracture at the bottom bolt. In this case, the ultimate tensile stress localizes around the tension zone while the shear stress in the shear zone between the two bolts does not achieve its ultimate value. Therefore, after the tensile fracture occurs at the bottom bolt in Region 3, shear yielding starts to occur in the shear zone between the two bolts in Region 2. This phenomenon occurs when the shear yielding strength ($R_y^S = 0.6f_y A_{av}$) in Region 2 between the two bolts is larger than or equal to the tensile fracture strength ($R^T = U_{sl}f_u A_{nt}$) in Region 3 (i.e., $R_y^{S(2)} \geq R^{T(3)}$). Besides, the active shear resistance planes of the bolted shear connections also occur during the testing.

The cold-formed steel design standard AS/NZS 4600:2018 provides a good correlation with the experimental results in general for the block shear capacity of the shear beam-end bolted connections because the shear fracture in Region 2 only occurs with the shear distance (s) relatively short. Furthermore, the difference between yield stress (f_y) and tensile strength (f_u) is relatively small by using high strength G450 and G500 steels with significantly lower ductility. Consequently, the difference between the shear yielding strength ($R_y^S = 0.6f_y A_{av}$) and the shear fracture strength ($R^T = U_{sl}f_u A_{nt}$) is reasonably insignificant. On the other hand, the AS/NZS 4600:2018 predictions are more conservative in obtaining the nominal block shear strength against the experimental results in specific cases where the edge distance (e_2) is relatively long while the shear distance (s) is relatively short. Therefore, the AS/NZS 4600:2018 predictions can be inaccurate and conservative to obtain the nominal block shear capacity without considering the effect of the shear distance (s) on the fracture process behaviour of the connections as observed from the experiments.

With a target reliability index of 3.5, the reliability analysis demonstrates that a resistance factor ($\phi = 0.90$) can be used for rupture of the bolted connections in high strength steel with lower ductility. This value is significantly higher than that employed by AS/NZS 4600:2018 and AISI S100:2016 (R2020) w/S3-22 due to the good quality control of the materials involved in the tests. As a result, a good testing control may increase a higher resistance factor (ϕ).

CHAPTER 5

EXPERIMENTAL INVESTIGATION OF BOLTED CONNECTIONS UNDER CURLING PHENOMENON

5.1. INTRODUCTION

This chapter demonstrates an effect of out of plane curling of the thin plate on the block shear and tear-out behaviour of high strength steel bolted connections between thin and thick plates based on an experimental program. A total of 48 tests were established with sheets of thickness of 1.0mm, 1.2mm and 1.9mm in conjunction with two different hole diameters (9mm and 14mm) and various end distances between the outer bolt holes and the free edge as the main variables. Three different failure modes being block shear, tear-out and curling were observed in the tests. In addition, different types of material properties of the cold-formed sheet steels (i.e., G300, G450, G500 and G550) were conducted in order to create a large database for the investigation. Consequently, a design equation for block shear and tear-out strength accounting for the curling influence is proposed. It provides a better correlation with the experimental results than those predicted in the current design standards and specifications from AS/NZS 4600:2018 [2] and AISI S100:2016 (R2020) w/S3-22 [1]. A reliability analysis is included in this chapter.

5.2. EXPERIMENTAL INVESTIGATION OF CURLING EFFECT ON BOLTED CONNECTIONS

The test rig was specially designed and built in the JW Roderick Structures Laboratory in the School of Civil Engineering, the University of Sydney. All test specimens were conducted using an MTS Sintech 65/G testing machine. In addition, testing methodologies are employed on the basis of the specifications from the AISI S905-13: Test standard for cold-formed steel connections [8], Specification for structural joints using ASTM A325 or A490 bolts [9], Research report RP01-4 – AISI: Testing of bolted cold-formed steel connections in bearing (with and without washers) [10], and the European recommendations – ECCS TC7 TWG 7.10: The testing of connections with mechanical fasteners in steel sheeting and sections [11].

Various grades of cold-formed steel sheets such as G300, G450, G500 and G550 to AS 1397-2021 [52] were used for the specimens to obtain a wide range of material properties. Furthermore, material properties of these steel sheets taking in the longitudinal direction were extracted from the coupon tests as demonstrated in Table 5.1. Details of the coupon tests also conducted at the University of Sydney are given in Appendix B. The cold-formed steel sheets with thicknesses of 1.0mm, 1.2mm and 1.9mm were cut in the longitudinal direction using a

waterjet cutting machine. High strength steel Grade 8.8 bolts were used to eliminate the shear failure in the bolts. Furthermore, two different diameters of the bolts (i.e., 8mm and 12mm) were employed in the tests to investigate the hole size effect on the curling load of the connections. In terms of the interaction between the bolts and the steel plates, a non-threaded section on the shank of the bolts was utilised to eliminate imperfection during the tests. Although the major failure mode was out-of-plane block shear failure with curling, the washer grade (i.e., M8×16mm×1.2mm stainless A2-70 G304 flat washers for M8 bolts and M12×24mm×2.5mm stainless A2-70 G304 flat washers for M12 bolts) was significantly less than the main CFS sheets. Furthermore, in screwed and PAFs connections, washer diameter significantly affects capacity of the connections. However, in bolted connection, only the type of connection (e.g., with or without washer) affects bearing capacity rather than block shear capacity of the connection. As a result, the washers did not influence the test results in the thesis and analysis where block shear failure was the main failure mode.

Table 5.1: Material properties of steel sheets (full test data in Appendix B)

Steel	Nominal thickness (mm)	Base metal measured thickness (mm)	Yield stress f_y (MPa)	Ultimate strength f_u (MPa)	Young's modulus of elasticity E (MPa)
G300	1.90	1.89	296.86	362.34	202,922
G450	1.90	1.89	542.78	569.70	201,119
G500	1.20	1.18	551.29	577.98	202,878
G550	1.00	0.99	562.09	595.49	208,699

A typical test arrangement of the bolted connections is demonstrated in Figure 5.1. In particular, one inside 10mm hot-rolled mild steel plate was connected to two outside thin cold-formed steel sheets through the high strength steel bolts. The bolts were arranged in a manner as demonstrated in Figure 5.2 to create a symmetry between the two different connections so that the out of plane curling of the two outside thin plates of the connections can occur simultaneously. The lower grip of the MTS machine was in fixed position and played a crucial role to hold the bottom steel plates while the upper grip moving upwards created a tensile force based on the displacement-control technique with a rate of 0.2mm/min of the crosshead speed to ensure a quasi-static response. The load results from the MTS machine and the displacement from the linear variable displacement transducers (LVDTs) were recorded simultaneously by a data logger. For displacement measurements, vertical LVDTs and horizontal LVDTs were placed at reasonable positions to measure respectively in-plane and out-of-plane displacement of the connections via the aluminium equal angles attached to the steel sheets, as can be seen in Figure

5.2. In addition, a high-resolution camera (i.e., Nikon D810 body and a 105 mm lens) was used during the experimental process to capture images in the area of interest.

Three different types of configurations of the bolted connections as illustrated in Figure 5.3 and Table 5.2 were used to investigate the curling effect on the block shear and tear-out strength and behaviour of the connections. Configuration Type 1 includes only 1 hole to evaluate the curling effect of one hole on the shear strength behaviour of the connections whereas the effect of hole arrangement on the curling behaviour of the connections is investigated in Configuration Types 2 and 3. The gauge distances (g) in Configuration Types 2 and 3 were chosen in order to create the block shear failure in the connections instead of bearing failure which was not of interest to investigate in this research although the gauge distance ($g = 3d_f$) is recommended by AISI S100:2016 (R2020) w/S3-22 [1]. Reduced values of g have ensured block shear failure and not tear-out failure. The effect of thickness and hole size on the curling behaviour of the bolted connections is also evaluated by considering three different thicknesses (i.e., 1.0mm, 1.2mm and 1.9mm) and two hole diameters (i.e., 9mm and 14mm).

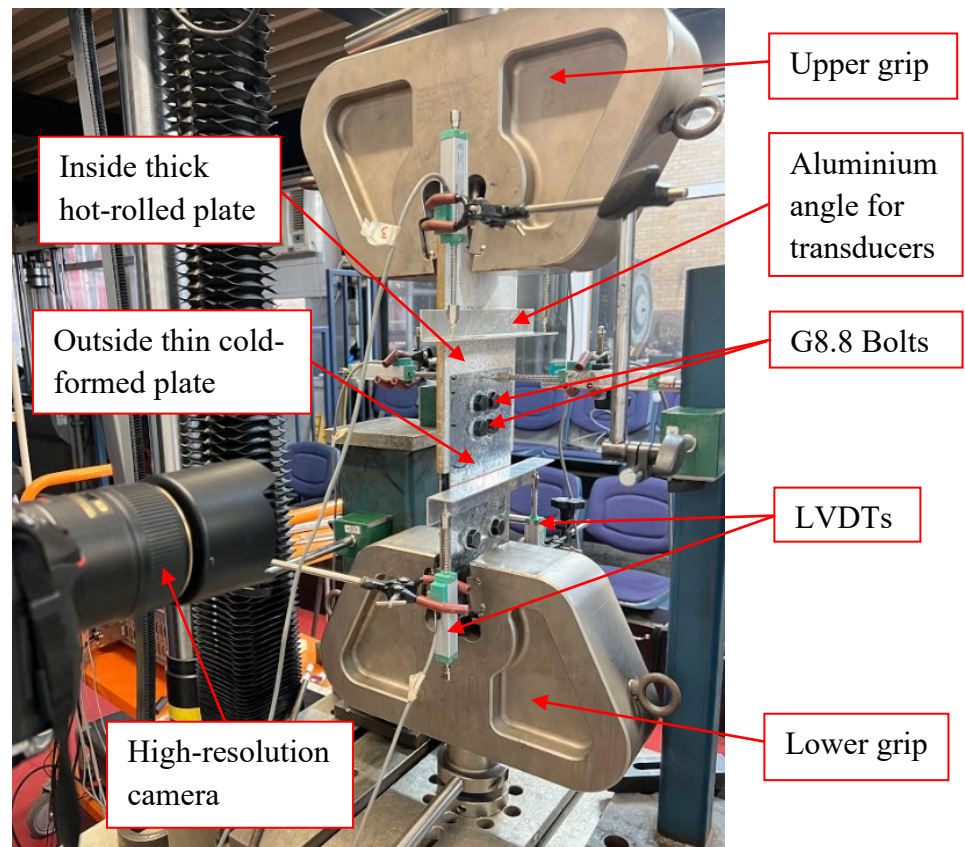


Figure 5.1: Test set-up of a bolted connection

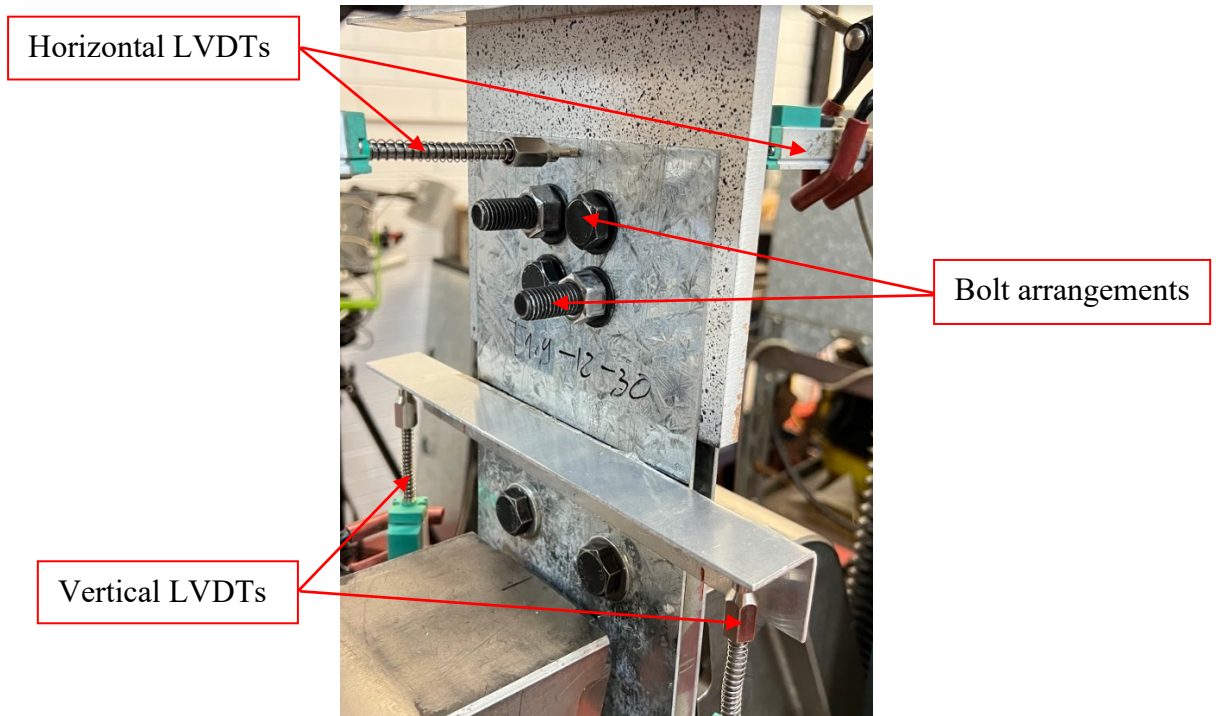


Figure 5.2: Schematic of a typical test specimen of a bolted connection

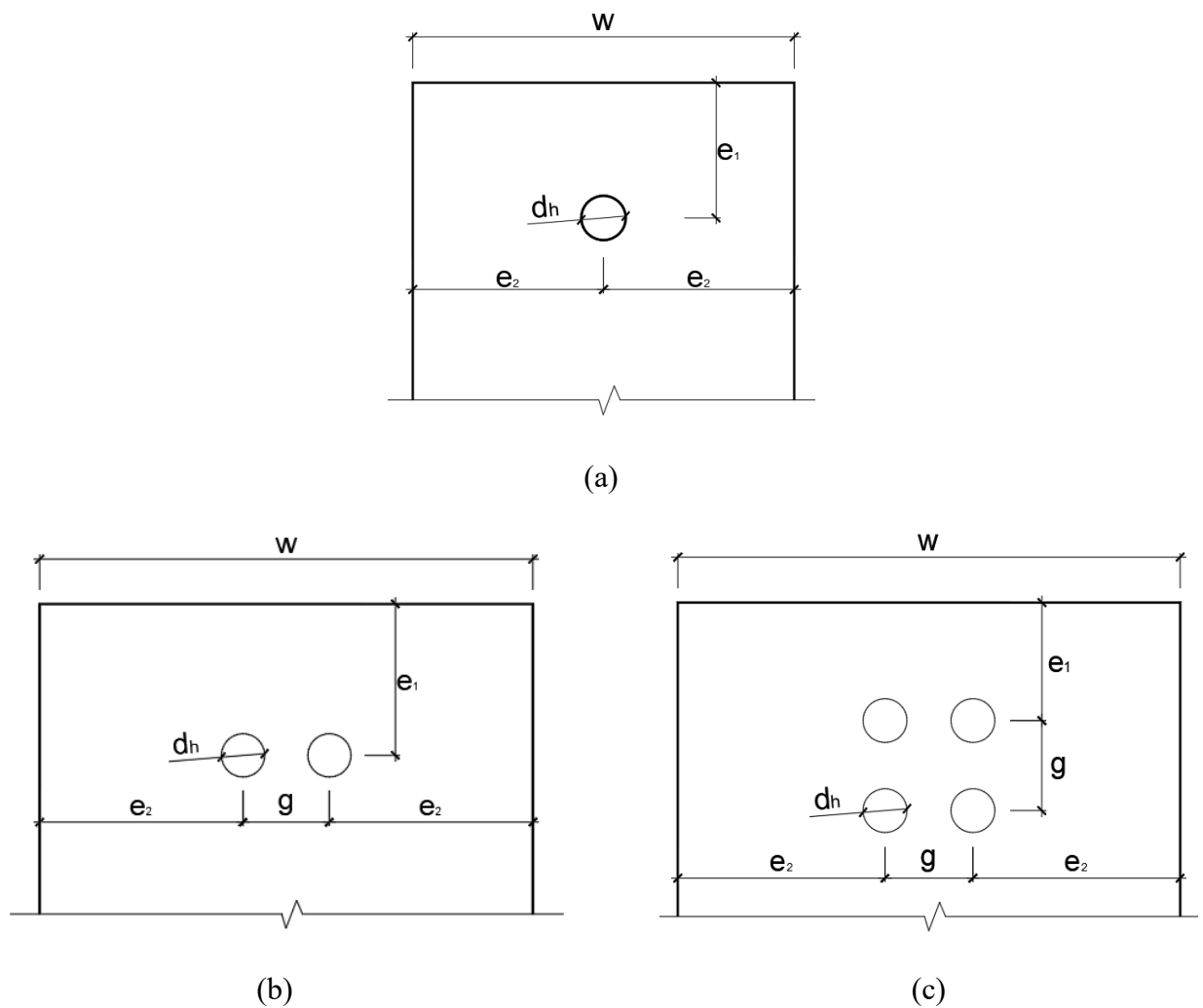


Figure 5.3: Schematic of bolted connections (a) Configuration Type 1, (b) Configuration Type 2 and (c) Configuration Type 3

Table 5.2: Specimen configurations

Config.	Specimen	Nominal thickness t (mm)	Nominal width w (mm)	Nominal hole size d_h (mm)	End distance e_1 (mm)	Edge distance e_2 (mm)	g (mm)
Type 1	T1.0-08-15-1H	1.00	120	9	15	60	-
Type 1	T1.0-08-18-1H	1.00	120	9	18	60	-
Type 1	T1.0-08-25-1H	1.00	120	9	25	60	-
Type 1	T1.0-12-19-1H	1.00	140	14	19	70	-
Type 1	T1.0-12-22-1H	1.00	140	14	22	70	-
Type 1	T1.0-12-30-1H	1.00	140	14	30	70	-
Type 2	T1.2-08-17-2H	1.20	120	9	17	51	18
Type 2	T1.2-08-20-2H	1.20	120	9	20	51	18
Type 2	T1.2-08-25-2H	1.20	120	9	25	51	18
Type 2	T1.2-12-18-2H	1.20	140	14	18	56	28
Type 2	T1.2-12-22-2H	1.20	140	14	22	56	28
Type 2	T1.2-12-25-2H	1.20	140	14	25	56	28
Type 2	T1.2-12-30-2H	1.20	140	14	30	56	28
Type 3	T1.2-08-17-4H	1.20	120	9	17	51	18
Type 3	T1.2-08-20-4H	1.20	120	9	20	51	18
Type 3	T1.2-12-22-4H	1.20	140	14	22	56	28
Type 3	T1.2-12-25-4H	1.20	140	14	25	56	28
Type 3	T1.9-08-28-4H	1.90	120	9	28	51	18
Type 3	T1.9-08-32-4H	1.90	120	9	32	51	18
Type 3	T1.9-08-35-4H	1.90	120	9	35	51	18
Type 3	T1.9-08-40-4H	1.90	120	9	40	51	18
Type 3	T1.9-12-25-4H	1.90	140	14	25	56	28
Type 3	T1.9-12-35-4H	1.90	140	14	35	56	28
Type 3	T1.9-12-42-4H	1.90	140	14	42	56	28

5.3. EXPERIMENTAL RESULTS

This section describes the experimental observation and results of the bolted connection tests of Configuration Types 1, 2 and 3 with different thicknesses, hole sizes and end distances to investigate the effect of the curling behaviour on the shear strength and the failure modes of the bolted connections. Two tests were repeated for each Configuration Type. The dimensions are identical for the repeated tests, in which their test names have an added suffix (i.e., 01, 02, etc.). For example, all the test names T1.0-08-15-1H-01, T1.0-08-15-1H-02 have a similar configuration.

5.3.1. Bolted connections Configuration Type 1

Specimens were fabricated with thickness of 1.0mm and the end distances (e_1) as the main variable. It is apparent from Table 5.3 that all the test results are consistent. The specimens T1.0-08-15-1H-01 and T1.0-08-15-1H-02 with a relatively short end distance (i.e., 15mm) failed by typical tear-out fracture without the curling. Nevertheless, the specimens T1.0-08-18-1H-01, T1.0-08-18-1H-02, T1.0-08-25-1H-01 and T1.0-08-25-1H-02 with relatively long end distances (e_1) (i.e., 18mm and 25mm, respectively) demonstrated the curling phenomenon, which caused a sudden strength reduction on the load-displacement curves as given in Figure 5.4a. In particular in Table 5.3, the shear strength of the connections increased by roughly 20% between $e_1 = 15mm$ and $e_1 = 18mm$ while it remained unchanged with $e_1 = 25mm$ due to the curling.

Table 5.3: Results of bolted connections Configuration Type 1

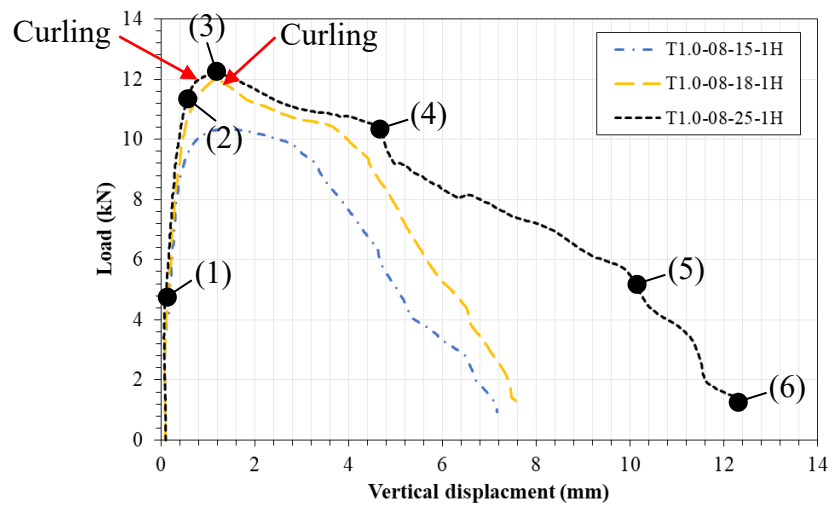
Steel	Specimen	Nominal thickness t (mm)	Nominal hole size d_h (mm)	End distance e_1 (mm)	Test results P^T (kN)	Limit state
G550	T1.0-08-15-1H-01	1.00	9	15	10.36	T
G550	T1.0-08-15-1H-02	1.00	9	15	10.43	T
G550	T1.0-08-18-1H-01	1.00	9	18	12.50	C
G550	T1.0-08-18-1H-02	1.00	9	18	11.96	C
G550	T1.0-08-25-1H-01	1.00	9	25	12.47	C
G550	T1.0-08-25-1H-02	1.00	9	25	12.22	C
G550	T1.0-12-19-1H-01	1.00	14	19	12.96	T
G550	T1.0-12-19-1H-02	1.00	14	19	12.58	T
G550	T1.0-12-22-1H-01	1.00	14	22	14.48	C
G550	T1.0-12-22-1H-02	1.00	14	22	14.76	C
G550	T1.0-12-30-1H-01	1.00	14	30	13.97	C
G550	T1.0-12-30-1H-02	1.00	14	30	14.08	C

Note: T – Tear-out failure; C – Curling failure

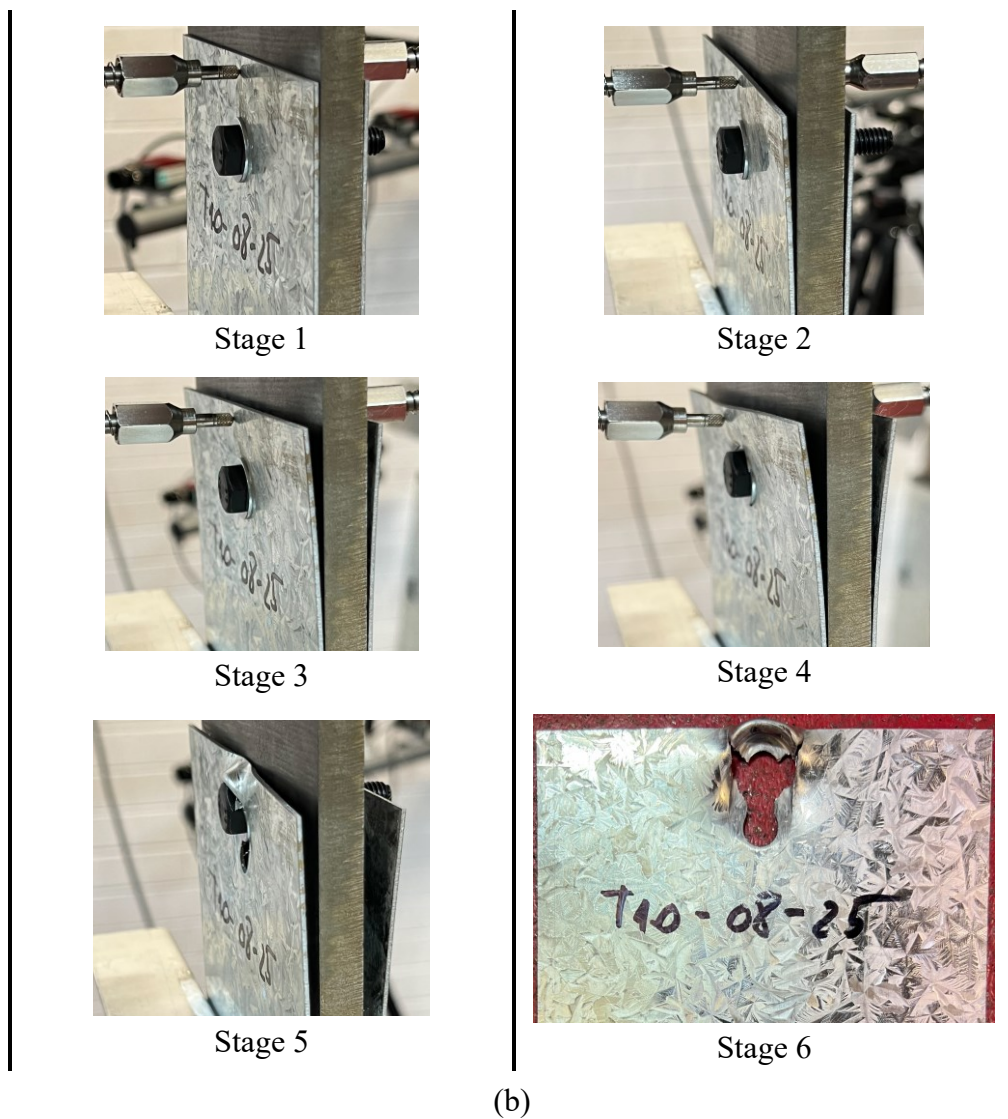
Similar behaviour was observed for the bolted connections with M12-high strength steel bolts. The shear strength of the connections also increased by 15% from $e_1 = 19mm$ to $e_1 = 22mm$ before remaining unchanged for a relatively long end distance ($e_1 = 30mm$). Consequently, the curling behaviour affects the shear strength of the bolted connections where the shear strength achieved an ultimate value and kept constant although the end distance still increased. In addition, the curling phenomenon is dependent on the hole size of the connections where the larger hole size (14mm) is less likely to curl than the smaller hole size (9mm).

Further, the load-displacement behaviour of the bolted connections using M8-bolts with various end distances are illustrated in Figure 5.4a. Clearly, the curling not only changed the behaviour but also affected the ultimate shear strength of the bolted connections. Especially, in the case of a relatively short end distance ($e_1 = 15mm$), a typical tear-out failure was observed whereas the curling effect was demonstrated for the remaining cases with relatively long end distances ($e_1 = 18mm; 25mm$) where a sudden strength reduction occurred after the ultimate load. The fracture process for the end distance $e_1 = 25mm$ is discretized into six stages in Figure 5.4a & b. The connection in this case is the most ductile although the ultimate shear strength keeps constant due to the curling. The shear strength of the connection increased gradually in the elastic phase (e.g., Stage 1) before the curling appeared at the free edge (e.g., Stage 2). In this stage, the out-of-plane displacement increased significantly in conjunction with a sudden strength reduction until the shear strength achieved an ultimate value (e.g., Stage 3). After that, the connection experienced a large extension in both in-plane and out-of-plane displacements (e.g., Stage 4) before fracture occurred in the shear areas (e.g., Stages 5 & 6). Similar behaviour was also observed in the case of the test with $e_1 = 18mm$ although it is less ductile than the previous one. This behaviour demonstrating the curling effect on bolted connections is also demonstrated in the studies by Kim et al. [3] and Kim and Kuwamura [4].

The behaviour of this Configuration Type was further studied using the M12-high strength steel bolts to investigate the effect of hole size on the shear strength of the connections under the presence of the curling, as can be seen in Figure 5.5. The behaviour of the configurations with larger hole sizes has the same trend as observed in Figure 5.4a. However, the ductility of the connection with the largest end distance ($e_1 = 30mm$) is slightly higher than that in the remaining cases ($e_1 = 19mm; 22mm$) instead of almost double as shown in Figure 5.4a. Details of the full set of load-displacement curves are given in Appendix D.



(a)



(b)

Figure 5.4: (a) Test results of 1.0mm cold-formed steel bolted connections Configuration Type 1 using M8-bolts and (b) Fracture process of the specimen T1.0-08-25-1H

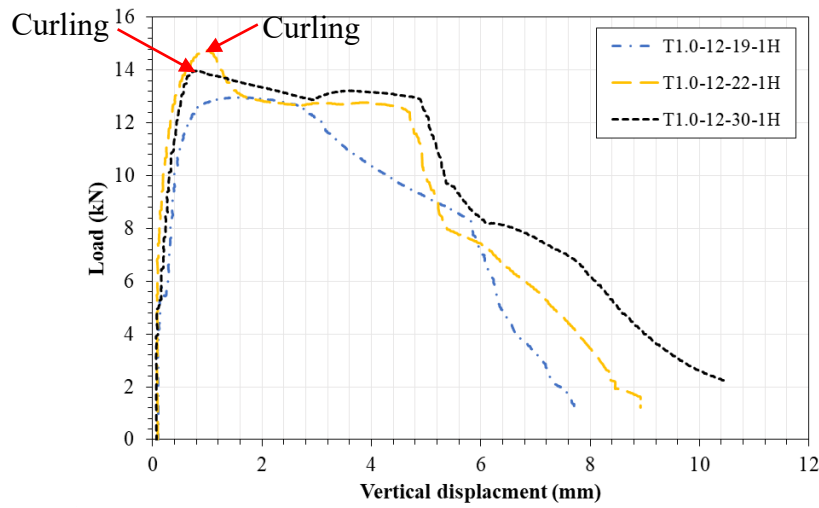


Figure 5.5: Test results of 1.0mm cold-formed steel bolted connections Configuration Type 1 using M12-bolts

5.3.2. Bolted connections Configuration Type 2

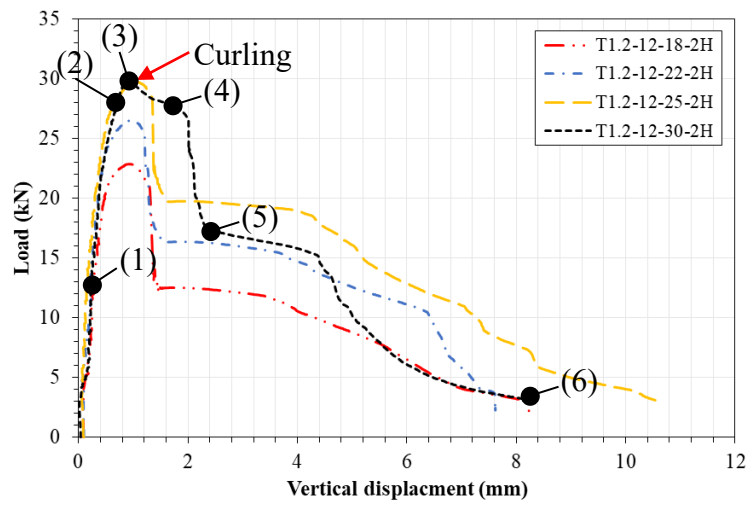
By following the same experimental procedure, the behaviour of the bolted connections of Configuration Type 2 was further examined to investigate the effect of hole arrangement on the curling behaviour of the connections. In this Configuration Type, specimens were fabricated with thickness of 1.2 mm with the presence of 2 holes in the cold-formed sheet steels. The test results are shown in Table 5.4. Again, the curling phenomenon also affected the shear strength of the connections in both cases of different hole sizes (i.e., 14mm and 9mm). The shear strength of the bolted connections using M12-bolts increased gradually by approximately 30% from $e_1 = 18mm$ to $e_1 = 25mm$ and then kept constant at an ultimate value due to the presence of the curling although the end distance e_1 was relative long (i.e., 30mm). Likewise in the case of M8-bolts, the curling occurred in the test of $e_1 = 25mm$. It may also affect the shear strength of the connections with larger end distances (i.e., $e_1 > 25mm$) which was not conducted in the experiments. In addition, it is apparent that this Configuration Type is less likely to curl by using thicker steel plates in comparison with the connections of Configuration Type 1 (i.e., 1.2mm and 1.0mm, respectively).

Table 5.4: Results of bolted connections Configuration Type 2

Steel	Specimen	Nominal thickness t (mm)	Nominal hole size d_h (mm)	End distance e_1 (mm)	Test results P^T (kN)	Limit state
G500	T1.2-12-18-2H-01	1.20	14	18	22.94	B
G500	T1.2-12-18-2H-02	1.20	14	18	22.89	B
G500	T1.2-12-22-2H-01	1.20	14	22	26.48	B
G500	T1.2-12-22-2H-02	1.20	14	22	25.55	B
G500	T1.2-12-25-2H-01	1.20	14	25	29.70	B
G500	T1.2-12-25-2H-02	1.20	14	25	28.04	B
G500	T1.2-12-30-2H-01	1.20	14	30	29.89	C
G500	T1.2-12-30-2H-02	1.20	14	30	30.77	C
G500	T1.2-08-17-2H-01	1.20	9	17	20.55	B
G500	T1.2-08-17-2H-02	1.20	9	17	20.38	B
G500	T1.2-08-20-2H-01	1.20	9	20	22.80	B
G500	T1.2-08-20-2H-02	1.20	9	20	22.29	B
G500	T1.2-08-25-2H-01	1.20	9	25	27.07	C
G500	T1.2-08-25-2H-02	1.20	9	25	27.13	C

Note: B – Block shear failure; C – Curling failure

Figure 5.6a illustrates the relationship between the vertical displacement and the block shear strength of the bolted connections of Configuration Type 2 using M12-bolts. In this particular Configuration Type, typical block shear failure was observed in the cases of relatively short end distances (i.e., 18 mm, 22 mm, 25 mm) while a sudden strength reduction on load-displacement curve due to the curling was demonstrated in the remaining case ($e_1 = 30\text{mm}$). In Figure 5.6b, the fracture process of the test of $e_1 = 30\text{mm}$ included six major stages. After the shear strength increased significantly in the elastic phase (e.g., Stage 1), the curling appeared at the free edge (e.g., Stage 2). In this step, the shear strength reached the peak (e.g., Stage 3) in a short time in conjunction with the increment of the out-of-plane displacement, followed by a sudden strength reduction (e.g., Stage 4). After that, the shear strength dropped dramatically (e.g., Stage 5) due to the tensile fracture between the two bolts in the critical area before the shear failure occurred in the remaining areas (e.g., Stage 6). During this stage, the connection underwent a large extension in both in-plane and out-of-plane displacements. Similar behaviour was observed in the connection using M8-bolts with $e_1 = 25\text{mm}$ as shown in Figure 5.7. Last but not least, the drop in the shear strength of the bolted connections Configuration Type 2 is much more severe than that of the bolted connections Configuration Type 1. Details of full set of load-displacement curves are given in Appendix D.



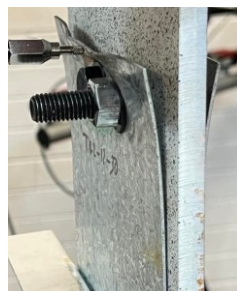
(a)



Stage 1



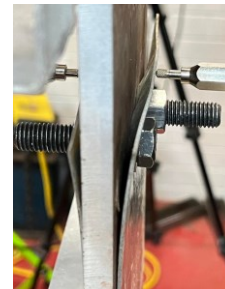
Stage 3



Stage 5



Stage 2



Stage 4



Stage 6

(b)

Figure 5.6: (a) Test results of 1.2mm cold-formed steel bolted connections Configuration Type 2 using M12-bolts and (b) Fracture process of the specimen T1.2-12-30-2H

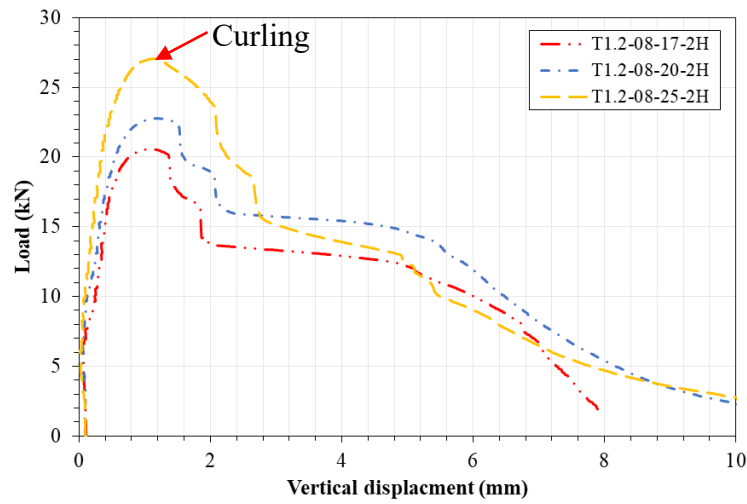


Figure 5.7: Test results of 1.2mm cold-formed steel bolted connections Configuration Type 2 using M8-bolts

5.3.3. Bolted connections Configuration Type 3

In this Configuration Type, 4 bolts and the steel plates with different grades of cold-formed steel sheets (i.e., G300, G450 and G500) were used not only to further investigate the effect of hole arrangement but also to examine the influence of material properties on the curling behaviour of the connections. Specimens were assembled with two sheets of thickness of 1.2mm and 1.9mm with various end distances based on the same previous experimental procedure.

The experimental results of the bolted connections of Configuration Type 3 are given in Table 5.5. For test specimens with the thickness of 1.2mm, typical block shear failure without the curling occurred for specimens with relatively short end distances. However, the curling effect was observed in the case of the thicker steel plate ($t = 1.9mm$) with relatively long end distances although lower grades of steel were used (i.e., G300 and G450). It implies that the curling behaviour is also dependent on both the thickness and grade of steel of the cold-formed steel sheets. In the special case of mild steel (i.e., G300), the shear strength increased by approximately 17% from $e_1 = 25mm$ to $e_1 = 35mm$ before remaining unchanged at an ultimate value due to the curling. This behaviour is similar to that observed in the previous configurations (i.e., Configuration Types 1 and 2).

Further, the relationship between load and displacement of the bolted connections of Configuration Type 3 is shown in Figure 5.8 - Figure 5.11. Based on the property of the mild steel, the specimens using G300 steel sheets illustrate a higher ductility (Figure 5.10a) in comparison with the others. In addition in Figure 5.10a & Figure 5.11, the curling behaviour was demonstrated through the sudden strength reduction on the load-displacement curves. Details of the full set of load-displacement curves are given in Appendix D.

Table 5.5: Results of bolted connections Configuration Type 3

Steel	Specimen	Nominal thickness t (mm)	Nominal hole size d_h (mm)	End distance e_1 (mm)	Test results P^T (kN)	Limit state
G500	T1.2-12-22-4H-01	1.20	14	22	40.72	B
G500	T1.2-12-22-4H-02	1.20	14	22	42.13	B
G500	T1.2-12-25-4H-01	1.20	14	25	43.15	B
G500	T1.2-12-25-4H-02	1.20	14	25	43.40	B
G500	T1.2-08-17-4H-01	1.20	9	17	34.42	B
G500	T1.2-08-17-4H-02	1.20	9	17	34.86	B
G500	T1.2-08-20-4H-01	1.20	9	20	37.51	B
G500	T1.2-08-20-4H-02	1.20	9	20	39.37	B
G300	T1.9-12-25-4H-01	1.90	14	25	42.13	B
G300	T1.9-12-25-4H-02	1.90	14	25	41.47	B
G300	T1.9-12-35-4H-01	1.90	14	35	49.14	C
G300	T1.9-12-35-4H-02	1.90	14	35	48.26	C
G300	T1.9-12-42-4H-01	1.90	14	42	49.88	C
G300	T1.9-12-42-4H-02	1.90	14	42	51.16	C
G450	T1.9-08-28-4H-01	1.90	9	28	59.39	B
G450	T1.9-08-28-4H-02	1.90	9	28	59.39	B
G450	T1.9-08-32-4H-01	1.90	9	32	64.49	C
G450	T1.9-08-32-4H-02	1.90	9	32	64.99	C
G450	T1.9-08-35-4H-01	1.90	9	35	64.46	C
G450	T1.9-08-35-4H-02	1.90	9	35	64.40	C
G450	T1.9-08-40-4H-01	1.90	9	40	63.83	C
G450	T1.9-08-40-4H-02	1.90	9	40	62.88	C

Note: B – Block shear failure; C – Curling failure

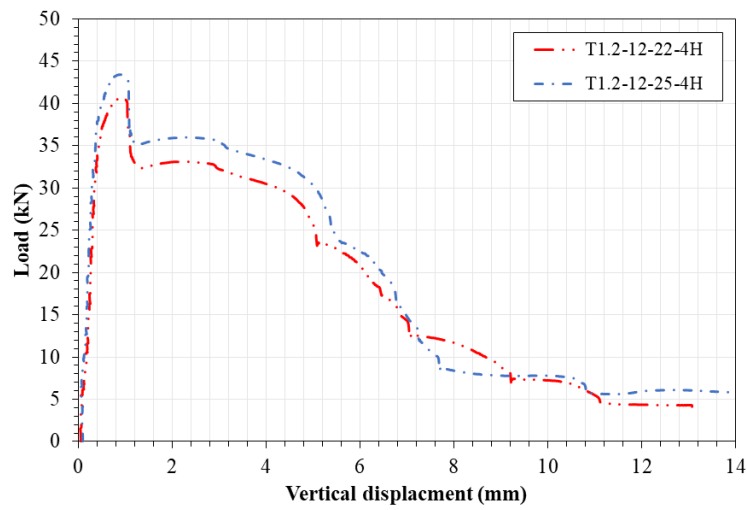


Figure 5.8: Test results of 1.2mm cold-formed steel bolted connections Configuration Type 3 using M12-bolts

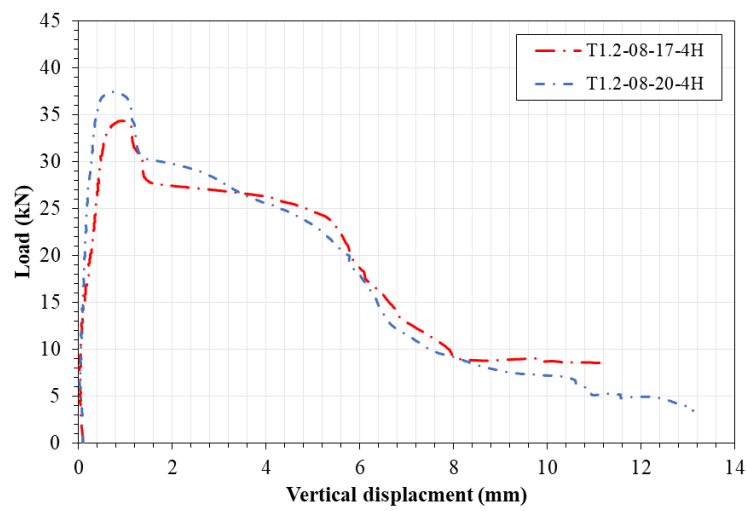
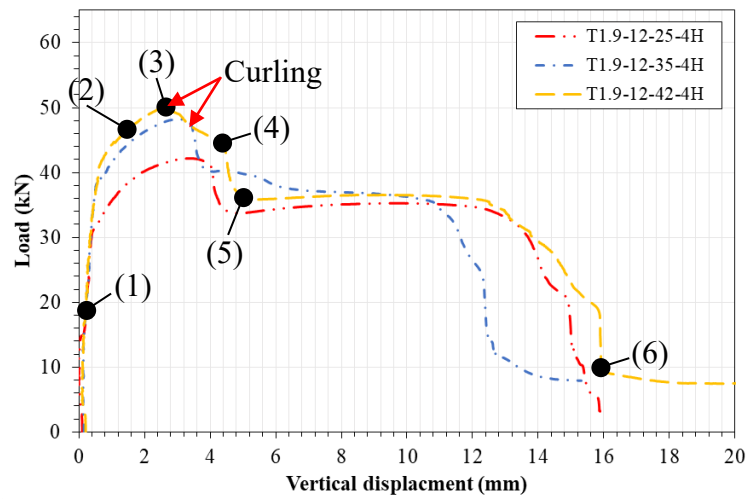


Figure 5.9: Test results of 1.2mm cold-formed steel bolted connections Configuration Type 3 using M8-bolts



(a)



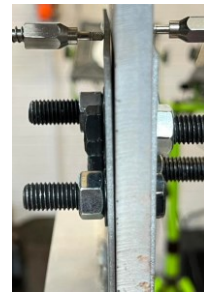
Stage 1



Stage 3



Stage 5



Stage 2



Stage 4



Stage 6

(b)

Figure 5.10: (a) Test results of 1.9mm cold-formed steel bolted connections Configuration Type 3 using M12-bolts and (b) Fracture process of the specimen T1.9-12-42-4H

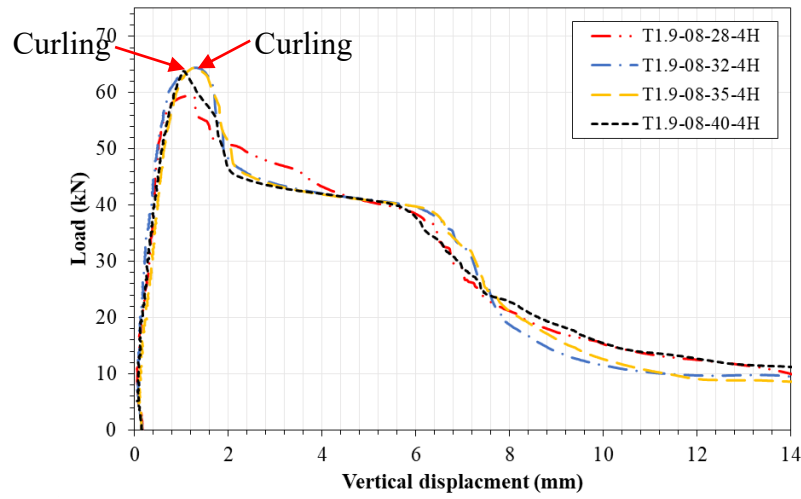


Figure 5.11: Test results of 1.9mm cold-formed steel bolted connections Configuration Type 3 using M8-bolts

5.4. DESIGN STANDARD AND SPECIFICATION COMPARISONS

5.4.1. Current design equations in standards/specifications

In the current cold-formed steel design standards and specifications from Australian and New Zealand (AS/NZS 4600:2018 [2]), Europe (Eurocode 3 Part 1.8. 2005 [15]) and the USA (AISI S100:16 (R2020) w/S3-22 [1]), the shear strength of the bolted connections is specified without the effect of the curling. For instance, the nominal block shear strength in AISI S100:16 (R2020) w/S3-22 – Section J6.3 [1] is determined as follows

$$R_n = 0.6f_y A_{av} + U_{bs} U_{sl} f_u A_{nt} \quad (5.1)$$

where R_n is the nominal shear strength of the connections; f_y and f_u denote yield stress, and tensile strength of the connected part, respectively; $A_{av} = (A_{gv} + A_{nv})/2$ is the active shear area lying in the middle of the net and gross shear areas; U_{bs} is a non-uniform block shear factor (0.5 for coped beam shear conditions with more than one vertical row of connectors, 1.0 for all other cases); U_{sl} is the shear lag factor;; A_{nt} is the net area subject to tension; A_{gv} is the gross area subject to shear; and A_{nv} is the net area subject to shear.

In the cold-formed steel design standard in Australia (AS/NZS 4600:2018 [2]), the design Equation (5.2) for block shear failure using an active shear area concept for the shear zones is similar to that in AISI S100:16 (R2020) w/S3-22 [1]. However, the design equation also does not consider the effect of the curling

$$R_n = 0.6f_y A_{av} + f_u A_{nt} \left(0.9 + 0.1 \frac{d_f}{s_f} \right) \quad (5.2)$$

in which A_{av} is the active shear area lying in the middle of the net and gross shear areas; the variable d_f denotes the bolt diameter; and s_f denotes the bolt spacing in the tensile resistance plane.

In addition in AS/NZS 4600:2018 [2], the design equation for determining tear-out strength of the bolted connections without the curling effect is specified as follows

$$V_f = t e_1 f_u \quad (5.3)$$

where V_f denotes the tear-out strength of the connections; and t is the thickness of the connected part.

Further, in Eurocode 3 Part 1.8 [15], the design equation for block shear failure without the effect of the curling is given in Equation (5.4)

$$R_n = \left(1/\sqrt{3} \right) f_y A_{nv} + f_u A_{nt} \quad (5.4)$$

5.4.2. Effect of the curling phenomenon on the stress distribution of the cold-formed plate

Based on both the experimental and calibrated FE results, the curling phenomenon not only affects the shear strength of the connections but also changes the stress distribution of the cold-formed plates. Under the effect of the curling, only a small area around the holes remains in-plane while the top area which is curled loses its shear strength (i.e., Figure 5.12). Moreover, under the action of force, the outstanding piece of plate undergoes an extension of the strain in the transverse direction (i.e., direction 1 in Figure 5.13a & Figure 5.14a) where the top area of the outstanding piece of plate is stretched while the bottom area is compressed as observed from the stress distribution in the transverse direction (S11). Therefore, the remaining outside areas of the plate play a role as a boundary condition to create transversely inverse stress acting back to the outstanding piece of plate. Finally, the curling is a direct consequence of the induced transverse compression in the outstanding piece of plate causing out of plane bending.

Consequently, a new proposal for evaluating the curling effect on the outstanding piece of plate caused by the bending is shown in Figure 5.13b & Figure 5.14b. The proposal implies that only the resistant areas contribute to the shear strength of the connections whereas the curled areas do not contribute to the shear strength of the connections resulting in no improvement on the shear strength with longer end distances. Further, the transverse stress causing the bending in

the outstanding piece of plate is induced by the remaining outside areas. This is similar to that observed in the experiments and calibrated FE results in the previous sections.

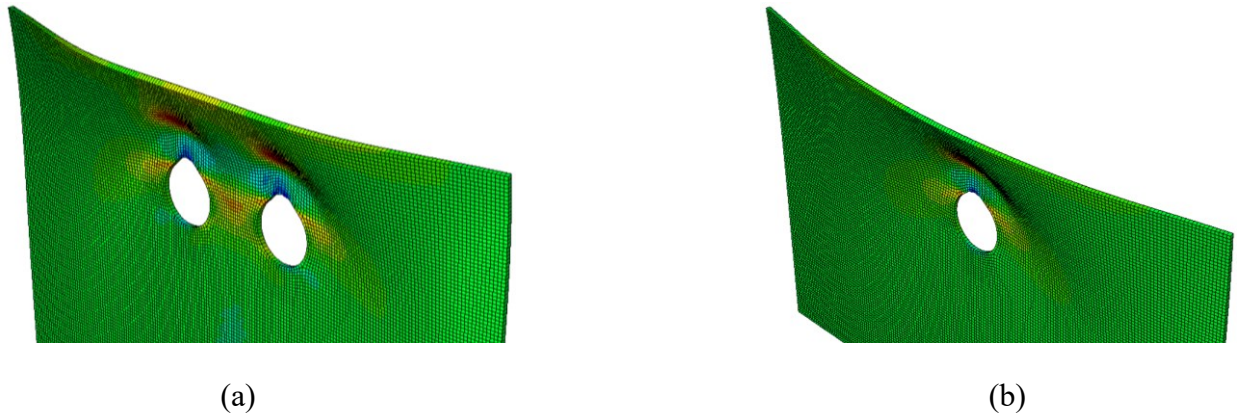


Figure 5.12: Curling behaviour of specimen (a) T1.2-12-30-2H (FEM) at Stage 4 (i.e., curling occurrence) and (b) T1.0-12-30-1H (FEM) at Stage 4 (i.e., curling occurrence)

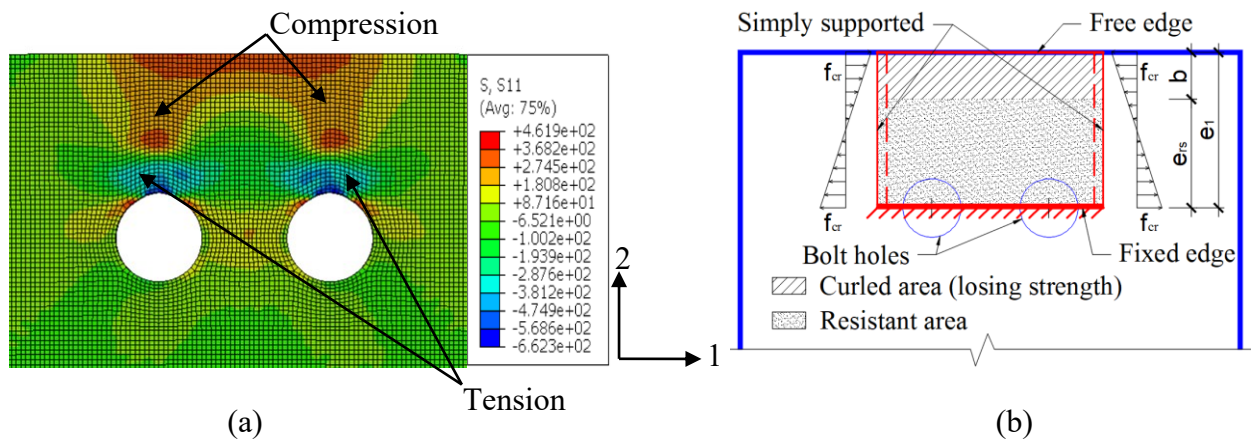


Figure 5.13: (a) Stress distribution in the transverse direction (S11) at Stage 3 (i.e., peak load) and (b) the corresponding virtual model of the outstanding piece of plate caused by the bending under the curling phenomenon of the specimen T1.2-12-30-2H (FEM)

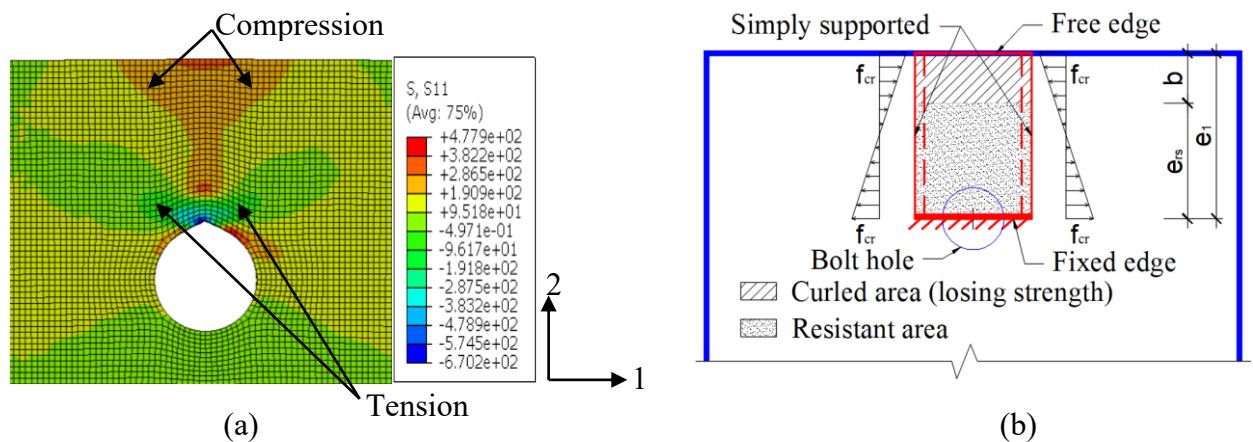


Figure 5.14: (a) Stress distribution in the transverse direction (S11) at Stage 3 (i.e., peak load) and (b) the corresponding virtual model of the outstanding piece of plate caused by the bending under the curling phenomenon of the specimen T1.0-12-30-1H (FEM)

Applying the buckling theory and effective width concept in AS/NZS 4600:2018 [2], the resistant length (e_{rs}) of the outstanding piece of plate is determined as follows

$$\text{If } \lambda_c \leq 0.673, e_{rs} = e_1 \quad (5.5)$$

$$\text{If } \lambda_c > 0.673, e_{rs} = \rho e_1 \quad (5.6)$$

where the effective width factor $\rho = (1 - 0.22/\lambda)/\lambda$; the slenderness ratio $\lambda = \sqrt{f_y/f_{cr}}$ and the plate elastic buckling stress (f_{cr}) can be determined from Equation (5.7)

$$f_{cr} = \frac{k\pi^2 Et^2}{12(1-\nu^2)(e_1)^2} \quad (5.7)$$

in which E is the Young's modulus of elasticity (200Gpa); ν is the Poisson's ratio (0.3); and the buckling coefficient $k = 2.15$ for plate simply supported on two sides with the top edge free and the bottom edge fixed (based on the proposal for the plate buckling coefficients under bending studied by Brockenbrough and Johnston [58] and Bijlaard [59]).

Finally, a new design Equation (5.8) for calculating the shear strength of the bolted connections under the presence of the curling is proposed depending on the failure mode. Particularly, only the resistant length (e_{rs}) is used to determine the shear strength of the connections instead of the end distance (e_1) as using in AISI S100:16 (R2020) w/S3-22 [1] and AS/NZS 4600:2018 [2]. Therefore, the shear strength of the connections determined by the design Equation (5.8) will achieve an ultimate value and keeps constant although the end distance continues to increase. This behaviour is similar to that observed in the experiments results.

$$\begin{cases} \text{Tear-out failure: } R_n^{prop.} = te_{rs}f_u \\ \text{Block shear failure: } R_n^{prop.} = 0.6f_yA_{av}^{rs} + U_{bs}U_{sl}f_uA_{nt} \end{cases} \quad (5.8)$$

where the active shear area (A_{av}^{rs}) is computed based on the resistant length (e_{rs}). The value of e_{rs} is limited at this stage to the range of geometric parameters used ($t = 1.0, 1.2, 1.9mm$), ($d_f = 8, 12mm$) but is believed to be general.

5.4.3. Proposed design equation

The accuracy of the proposed Equation (5.8) is confirmed in comparison with the experimental results as illustrated in Table 5.6. The ratio of the average test results divided by the predicted values ranges from 0.93 to 1.27. It is worth pointing out that the proposed formula (5.8) provides

a good agreement with the experimental results via the average ratios $P^T/R_n^{\text{Prop.}}$ and the corresponding coefficient of variation (i.e., Mean = 1.11 and C.o.V = 7.92%).

Table 5.6: Comparison of the experimental results with the proposed design Equation (5.8) for the cold-formed steel bolted connections

Config.	Specimen	Nominal thickness t (mm)	Nominal hole size d_h (mm)	End distance e_1 (mm)	Resistant length e_{rs} (mm)	Test results P^T (kN)	Proposed formula $R_n^{\text{Prop.}}$ (kN)	Ratio $\frac{P^T}{R_n^{\text{Prop.}}}$
Type 1	T1.0-08-15-1H	1.00	9.0	15	15.0	10.40	8.93	1.16
Type 1	T1.0-08-18-1H	1.00	9.0	18	17.8	12.23	10.63	1.15
Type 1	T1.0-08-25-1H	1.00	9.0	25	20.2	12.35	12.04	1.03
Type 1	T1.0-12-19-1H	1.00	14.0	19	18.3	12.77	10.89	1.17
Type 1	T1.0-12-22-1H	1.00	14.0	22	19.4	14.62	11.54	1.27
Type 1	T1.0-12-30-1H	1.00	14.0	30	21.2	14.03	12.64	1.11
Type 2	T1.2-08-17-2H	1.20	9.0	17	17.0	20.47	17.60	1.16
Type 2	T1.2-08-20-2H	1.20	9.0	20	20.0	22.55	19.99	1.13
Type 2	T1.2-08-25-2H	1.20	9.0	25	22.9	27.10	22.31	1.21
Type 2	T1.2-12-18-2H	1.20	14.0	18	18.0	22.92	20.67	1.11
Type 2	T1.2-12-22-2H	1.20	14.0	22	21.7	26.02	23.61	1.10
Type 2	T1.2-12-25-2H	1.20	14.0	25	22.9	28.87	24.58	1.17
Type 2	T1.2-12-30-2H	1.20	14.0	30	24.4	30.33	25.76	1.18
Type 3	T1.2-08-17-4H	1.20	9.0	17	17.0	34.64	28.32	1.22
Type 3	T1.2-08-20-4H	1.20	9.0	20	20.0	38.44	30.70	1.25
Type 3	T1.2-12-22-4H	1.20	14.0	22	21.7	41.43	40.28	1.03
Type 3	T1.2-12-25-4H	1.20	14.0	25	22.9	43.28	41.25	1.05
Type 3	T1.9-08-28-4H	1.90	9.0	28	28.0	59.39	57.77	1.03
Type 3	T1.9-08-32-4H	1.90	9.0	32	32.0	64.74	62.72	1.03
Type 3	T1.9-08-35-4H	1.90	9.0	35	34.6	64.43	65.93	0.98
Type 3	T1.9-08-40-4H	1.90	9.0	40	36.6	63.36	68.45	0.93
Type 3	T1.9-12-25-4H	1.90	14.0	25	25.0	41.80	37.85	1.10
Type 3	T1.9-12-35-4H	1.90	14.0	35	35.0	48.70	44.62	1.09
Type 3	T1.9-12-42-4H	1.90	14.0	42	42.0	50.52	49.36	1.02
Mean								1.11
C.o.V (%)								7.92

5.4.4. Reliability analysis

The reliability of the proposed formula is investigated in this section based on the test results of the bolted connections failing in the block shear and tear-out modes under the effect of curling so that the reliability indices (i.e., resistance factor ϕ and safety factor Ω) can be calibrated. Details of the reliability procedure was discussed previously in Chapter 3 where the resistance factor ϕ and safety factor Ω are determined as follows

$$\phi = \left(\frac{1.2 D/L + 1.5}{1.05 D/L + 1} \right) (P_m M_m F_m) \exp \left(-\beta \sqrt{C_P V_P^2 + V_M^2 + V_F^2 + V_Q^2} \right) \quad (5.9)$$

$$\text{and } \Omega = \frac{1}{\phi} \left(\frac{1.2 D/L + 1.5}{D/L + 1} \right) \quad (5.10)$$

In the above equations, the value of P_m is the mean value of the test-to-prediction ratio and the corresponding coefficient of variation (V_p) which were derived from the tests. Therefore, $P_m = 1.11$ and $V_p = 0.0792$ as can be seen in Table 5.6. Remaining variables were calculated in detail in Chapter 3.

Figure 5.15 illustrates the relationship between the reliability index (β) and the ratio of dead load to live load (D/L) for various values of the resistance factor (ϕ). Again, a lower value of the resistance factor (ϕ) demonstrates a higher index of the reliability (β). On the other hand, a higher value of the reliability index (β) can be achieved when using a higher value of the ratio of (D/L). This behaviour is similar to that observed in Chapter 3 and Chapter 4.

Furthermore, the ratio D/L and the target reliability index (β) are respectively taken as 0.2 and 3.5 for the purposes of checking the reliability based on LRFD criteria. The ratio ($D/L = 0.2$) is typical for light weight CFS structures while the target reliability index ($\beta = 3.5$) is chosen to eliminate the initial failure in the connections. Table 5.7 demonstrates the values of the resistance factors ($\phi = 0.86$) and safety factors ($\Omega = 1.68$) of the proposed formula for rupture of the bolted connections with $D/L = 0.2$ and $\beta = 3.5$. Again, these reliability indices (i.e., $\phi = 0.86$ and $\Omega = 1.68$) reflect a good quality control of the materials involved in the tests and the accuracy of the proposed formulae in comparison with the CFS Specification from AISI S100:2016 (R2020) w/S3-22 [1] (i.e., $\phi = 0.75$ and $\Omega = 2.00$).

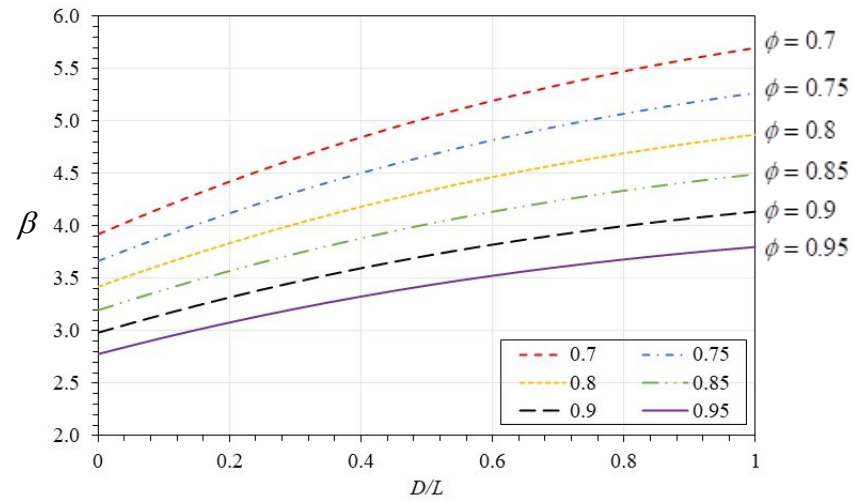


Figure 5.15: Reliability index β with ratio of dead load to live load D/L for different values of the resistance factor ϕ

Table 5.7: Resistance factor ϕ and safety factor Ω of the proposed formula with $D/L = 0.2$ and $\beta = 3.5$

Information		Resistance factor ϕ	Safety factor Ω
Mean	$P_m = 1.11$		
C.o.V	$V_p = 0.0792$	0.86 (0.75)*	1.68 (2.00)*
Number of samples	$n_T = 24$		

Note: * Values obtained from the AISI S100:2016 (R2020) w/S3-22 [1] Specification

5.5. SUMMARY

This chapter presents a total of 48 tests under quasi-static monotonic loading to investigate the complex behaviour of the bolted connections failing in the block shear and tear-out modes under the effect of curling phenomenon. The curling not only affected the shear strength but also changed the behaviour of the bolted connections. Bolted connections failed by a typical tear-out or block shear fracture without curling when the end distances were relatively short while the bolted connections which demonstrated curling in the cases of the relatively long end distances caused a sudden strength reduction in the load-displacement curves. The shear strength achieved an ultimate value and kept constant although the end distance continued to increase. As investigated, the curling behaviour is dependent on the hole size, thickness and grade of steel of the cold-formed plates of the connections where larger hole sizes, thicker plates and higher grades of steel are less likely to curl than smaller hole sizes, thinner plates and lower grades of steel, respectively.

Moreover, the curling behaviour also changed the stress distribution of the cold-formed plates observed from the experimental and calibrated FE results. Under the effect of the curling, only a small area around the holes remained in-plane whereas the top curled area loses the shear strength in the connection. In addition, the curling was a direct consequence of the induced transverse compression in the outstanding piece of plate causing bending. Consequently, a new design equation for the block shear and tear-out strength accounting for the curling effect is proposed. The proposal implies that only the resistant areas remain to contribute to the shear strength of the connections whereas the curled areas do not add to the shear strength of the connections resulting in no improvement on the shear strength with longer end distances. This is similar to that observed to the experimental results.

Comparison of the test results with the design standards and specifications from AS/NZS 4600:2018 and AISI S100:16 (R2020) w/S3-22 demonstrates that the current design equations in standards/specifications overestimate the nominal shear strength without the consideration of curling. Nevertheless, the proposed formula reflects very well the actual behaviour based on the buckling theory and effective width concept instead of adding an influence coefficient to represent the curling effect. Hence, the proposal provides an exact value of the end distance when the curling starts to occur in order to appropriately evaluate the shear strength of the bolted connections.

With a target reliability index (β) of 3.5, the reliability analysis demonstrates that the reliability indices (i.e., resistance factor $\phi = 0.86$ and safety factor $\Omega = 1.68$) under the effect of curling are better than the current values used for block shear and tear-out ruptures of the CFS bolted connections.

CHAPTER 6

FINITE ELEMENT MODELLING OF COLD-FORMED STEEL BOLTED CONNECTIONS

6.1. INTRODUCTION

This chapter illustrates the development of finite element models (FEM) using the ABAQUS software [14]. The finite element (FE) simulation is calibrated against the test results obtained from the above series of tests in order to provide additional insights which could not be seen in actual tests (i.e, stress and strain distributions). Furthermore, the calibrated FE models are used to generate a reliable data-set in a parametric study for the verification procedure of the proposed formulas over a wider range of data. In the numerical FE simulation, fracture criterion of the materials, loading rate, mesh sensitivity as major aspects of the FE model are calibrated and discussed to verify the accuracy of the FE model.

6.2. DEVELOPMENT OF THE FINITE ELEMENT (FE) MODELS

6.2.1. Selection of ABAQUS solver

Two complementary analysis tools provided in ABAQUS are ABAQUS/Standard and ABAQUS/Explicit (Systèmes 2014). In particular, the ABAQUS/Standard tool provides the capability to analyse static and linear dynamic problems where unknown values (at the current time) are obtained from the current information. Therefore, iteration and convergence checking procedures are required. Once convergence is achieved, however, the time increment can be large, and it is very time consuming. Nevertheless, the ABAQUS/Explicit tool uses a dynamic method which integrates the equations of motion to obtain unknown values where neither iteration nor convergence checking is required. Therefore, the ABAQUS/Explicit tool is suitable for large, nonlinear, quasi-static analyses and highly discontinuous post-buckling and collapse simulations. For this reason, the ABAQUS/Explicit tool is used for the simulations against the current test results in this thesis.

6.2.2. Contacts and interactions

Surface to surface contact (explicit) is used for the interaction between the steel sheets and between the bolt threads and the holes. The contact properties include two behaviours defined in ABAQUS: Normal Behaviour with separation after contact is allowed, and Tangential Behaviour with Penalty friction formulation. A friction coefficient of 0.3 between the sheet steels is used in the FEM simulation (Truong et al. [47]). In addition, the resistance between the bolt threads and

the holes is created from not only the friction but also the restraint when the threads of the bolts tapped into the holes. In order to ensure that the friction is large enough to hold the bolts in place, calibration had to be implemented. Consequently, the friction between the bolt threads and the holes is chosen as 1.2. In the simulation, the bolt model uses solid deformable elements with a non-threaded section on the shank. This 3D model is important to match the real behaviour of the bolts.

6.2.3. Element type and mesh size

In the ABAQUS/Explicit simulation for large, nonlinear, and quasi-static analyses, stable time increment (Δ_t) is one of the parameters, affecting the computing time. In particular, in Equation (6.1), the stable time increment (Δ_t) definitely depends on the change of the smallest characteristic length of the elements (L_e) in the entire model while the dilatational wave speed of the steel material (c_d) is a constant (i.e., approximately 6000m/s according to the ABAQUS Analysis User's Guide [60]). In addition, eight-node brick elements with reduced integration (C3D8R) are used for the entire model in the simulation. The selection of mesh size is calibrated based on the current test results in Chapters 3, 4 and 5.

$$\Delta_t = \frac{L_e}{c_d} \quad (6.1)$$

6.2.4. Selection of loading rate

In the experimental program, the rate of 0.2 mm/min of the crosshead speed was chosen to ensure a quasi-static response while, in the FEM simulation, a load speed of 210 mm/min is utilised not only to get a suitable response with insignificant effect of kinetic energy but also to reduce the computing time. It should be noted, however, that as the quasi-static material properties are the input to ABAQUS, the loading rate in the FEM does not affect results.

6.3. VERIFICATION of FE MODELS AGAINST THE EXISTING EXPERIMENTS

6.3.1. Staggered hole bolted connections

6.3.1.1 Input parameters

Figure 6.1 illustrates the Finite Element model of the staggered hole bolted connection by means of the ABAQUS/Explicit simulation. Material properties of the G450 steel sheets are employed from the coupon tests taken in the longitudinal and transverse directions as in Table 3.1 and in Figure 6.2. In addition, material properties of the G300 thick packing plates between CFS sheets are extracted from the coupon tests in Appendix B. Consequently, according to the ABAQUS

Analysis User's Guide [60], the data plays a crucial role as inputs into the ABAQUS simulations after taking the transformation of the nominal stress and strain to the true stress and true plastic strain. Figure 6.3 shows the FE input against the coupon tests in the longitudinal direction.

$$\sigma_{true} = \sigma_{nom} (1 + \varepsilon_{nom}) \quad (6.2)$$

$$\varepsilon_{true} = \ln(1 + \varepsilon_{nom}) - \frac{\sigma_{true}}{E} \quad (6.3)$$

where σ_{nom} and ε_{nom} are nominal stress and strain extracted from the coupon tests, respectively; σ_{true} and ε_{true} denote true stress and strain used in the ABAQUS simulations, respectively.

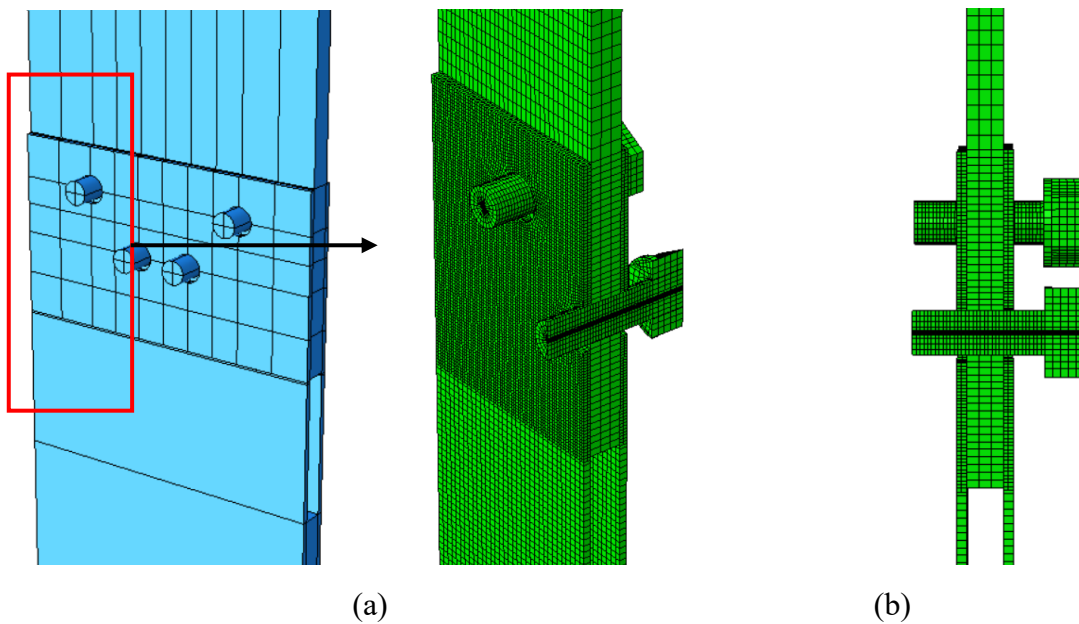


Figure 6.1: Finite Element Modelling of the staggered hole bolted connection: (a) the assembly and (b) the bolt in contact with the hole. Note: a load speed of 210 mm/min was applied to the top hot-rolled plate while fixed-end boundary condition was applied to the bottom hot-rolled plate, which was similar to the corresponding experimental program

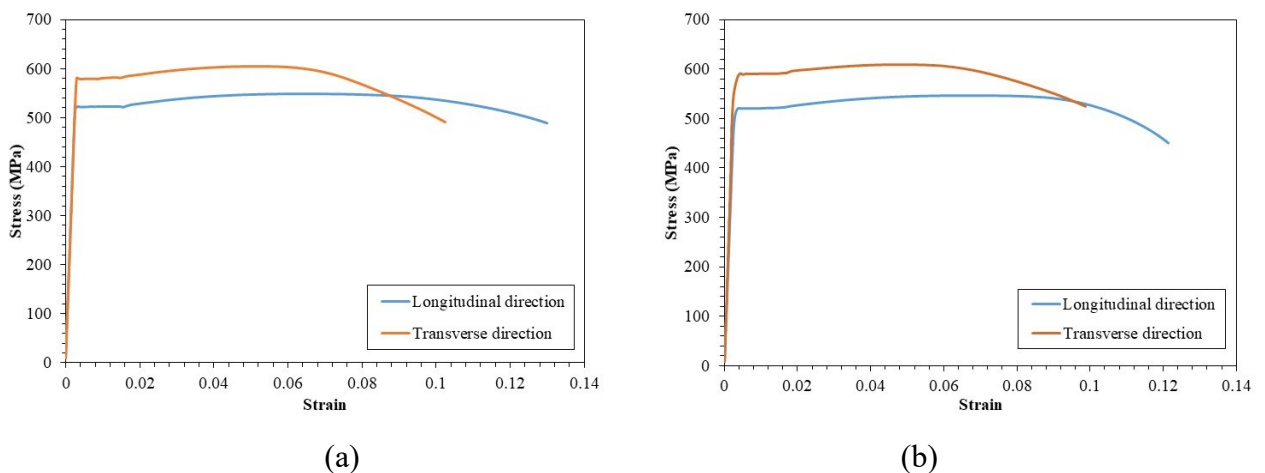


Figure 6.2: (a) Stress-strain curves of 1.9mm cold-formed sheets and (b) stress-strain curves of 2.4mm cold-formed sheets

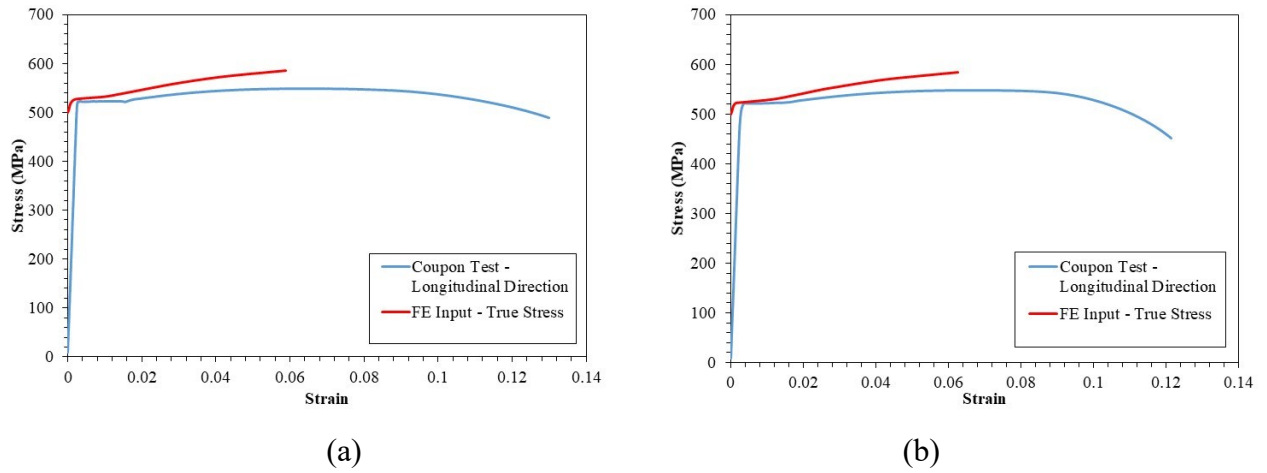


Figure 6.3: Stress-strain curves from coupon tests in longitudinal direction and FE input for (a) 1.9mm cold-formed sheets and (b) 2.4mm cold-formed sheets

Furthermore, in the ABAQUS simulation, a through thickness isotropic material behaviour (i.e., the plane through thickness direction is the plane of isotropy at every point in the material) is applied for the elastic material model that reflects approximately the behaviour of the steel sheets after experiencing a longitudinal rolling process. It is not an isotropic material property as for common steels. According to the ABAQUS Analysis User's Guide [60], transverse isotropy is a special subclass of orthotropy, which is characterised by a plane of isotropy at every point in the material. In particular, after the longitudinal rolling process, the shapes of the molecules in the through thickness direction are quite similar as those in the transverse direction, as can be seen in Figure 6.4. Therefore, the 1–2 plane (i.e., the plane which contains both the through thickness direction and the transverse direction) is assumed to be the plane of isotropy at every point. The longitudinal direction is the anisotropic direction.

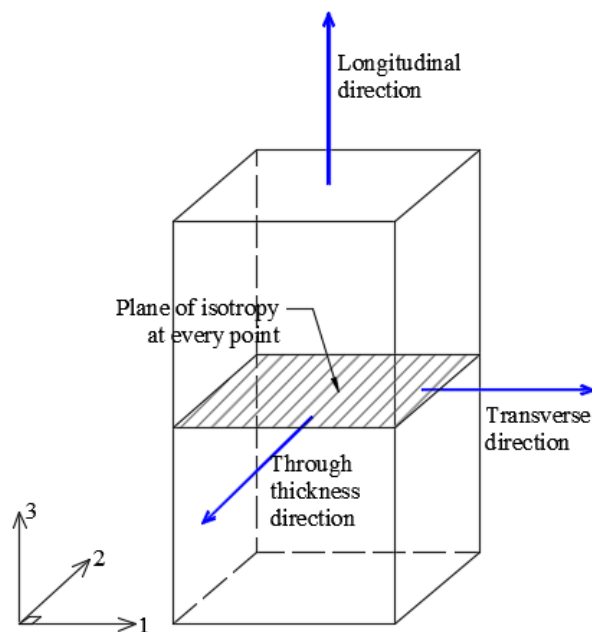


Figure 6.4: Schematic of the plane of isotropy at every point in a high strength cold-rolled steel sheet after experiencing a longitudinal rolling process.

The transversely isotropic material parameters are extracted from the experimental coupon tests in both the longitudinal and transverse directions to obtain the Young's Modulus, Poisson's ratio and Shear Modulus in the FEM, as can be seen in detail in Table 6.1. Consequently, Figure 6.5 and Table 6.2 provide the results using the transversely isotropic material property for high strength cold-formed steels in the FEM simulation as compared with the simulation results that use the isotropic material property. Both FE simulations are compared with current test results of the specimen A1.9-12-00. It appears that the anisotropy has little effect on the results with the longitudinal properties dominating.

Table 6.1: Material properties of cold-formed sheet steels in the FE simulation extracted from the coupon tests in the longitudinal and transverse directions of the staggered hole bolted connections

Steel	Young's Modulus $E_1 = E_2$ (MPa)	Young's Modulus E_3 (MPa)	Poisson's ratio ν_{12}	Poisson's ratio $\nu_{13} = \nu_{23}$	Shear Modulus G_{12} (MPa)	Shear Modulus $G_{13} = G_{23}$ (MPa)
G450	213681	199751	0.273	0.3	83928	82185

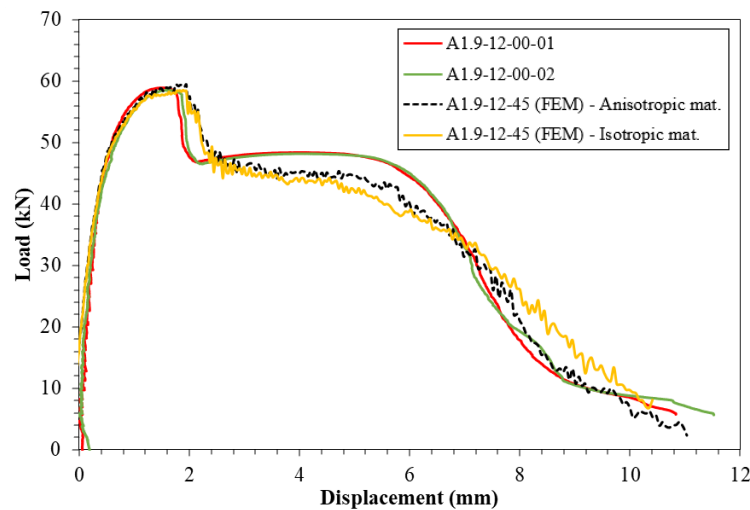


Figure 6.5: Comparison of the FE results using transversely isotropic material property and isotropic material property for high strength cold-formed steels against current test results of the specimen A1.9-12-00

Table 6.2: Comparison of the FE results using transversely isotropic material property and isotropic material property for high strength cold-formed steels against current test results of the specimen A1.9-12-00

Specimen	Test results P_1^T (kN)	Test results P_2^T (kN)	FE results $P_{Trans. Mat.}^{FE}$ (kN)	FE results $P_{Iso. Mat.}^{FE}$ (kN)	$\frac{P_{Trans. Mat.}^{FE}}{P_1^T}$	$\frac{P_{Iso. Mat.}^{FE}}{P_1^T}$	$\frac{P_{Trans. Mat.}^{FE}}{P_2^T}$	$\frac{P_{Iso. Mat.}^{FE}}{P_2^T}$
A1.9-12-00	58.94	58.56	59.46	59.64	1.009	1.012	1.015	1.018

Within this model, to simulate the plasticity phase for the transversely isotropic material behaviour, a Hill's plasticity model is employed using the coupon test results in the longitudinal direction as per the reference data. Therefore, a factor $R = f_y^{trans} / f_y^{long}$ as the ratio between the yield stress in the transverse (f_y^{trans}) and the longitudinal (f_y^{long}) directions is applied in the Hill's plasticity model. The detailed calculation method for the factors in Hill's plasticity model is explained by Wang et al. [61].

In addition, a “Ductile Damage” model is utilised for the fracture model in the ABAQUS simulation. Required parameters in this model include the fracture locus of the fracture strain versus the stress triaxiality and the strain rate. The effect of the stress triaxiality and equivalent strain to fracture on the ductile fracture of aluminum alloy was investigated by Bao and Wierzbicki [62]. In their study, the relationship between the equivalent strain and stress triaxiality was analysed via various tests where a wide range of stress triaxiality was performed (i.e., from 0 to 0.95). Consequently, they pointed out that the influence of the stress triaxiality must be established by three distinct regions instead of a monotonic function as studied by Johnson and Cook [63] and Choung et al. [64]. Particularly, shear fracture dominates in the range of negative stress triaxialities (i.e., from $-1/3$ to 0). Fracture occurs because of void formation which is in the range of high stress triaxialities (i.e., from 0.4 to 0.95), whereas in the range of medium stress triaxialities (i.e., from 0 to 0.4), fracture may appear as a result of a combination of shear and void growth mode, and is given by

$$\text{for } \sigma_H / \bar{\sigma} = -1/3 - 0: \bar{\varepsilon}_f = 0.1225 \left(\frac{\sigma_H}{\bar{\sigma}} + \frac{1}{3} \right)^{-0.46} \quad (6.4)$$

$$\text{for } \sigma_H / \bar{\sigma} = 0 - 0.4: \bar{\varepsilon}_f = 1.9 \left(\frac{\sigma_H}{\bar{\sigma}} \right)^2 - 0.18 \left(\frac{\sigma_H}{\bar{\sigma}} \right) + 0.21 \quad (6.5)$$

$$\text{for } \sigma_H / \bar{\sigma} = 0.4 - 0.95: \bar{\varepsilon}_f = 0.15 \left(\frac{\sigma_H}{\bar{\sigma}} \right)^{-1} \quad (6.6)$$

where σ_H and $\bar{\sigma}$ are hydrostatic stress and equivalent stress, respectively, $\bar{\varepsilon}_f$ is equivalent strain to fracture.

It is clear that the effect of the stress triaxiality and equivalent strain to fracture on the ductile fracture is not the same for different steels as studied by Bao and Wierzbicki [62], Bao [65] and Mirza et al. [66]. Therefore in this paper, a proposal of fracture ductile for high strength cold-formed steel is employed based on currently actual tests and proposed formulas from Bao and Wierzbicki [62] and Suárez et al. [67], as can be seen in Figure 6.7a. For instance, the test results of the specimen A2.4-12-00 are investigated for the calibration procedure. In particular,

different calibration curves (i.e., Calibration Nos. 1, 2 and 3) for the fracture model in Figure 6.6 are established to figure out the appropriate ductile fracture model for the high strength cold-formed steel. By means of fracture characteristics in each region (i.e., low, medium and high triaxialities), it is recommended to change the slope (lower fracture strain curve produces lower ductility) of the calibration curves. Furthermore, details of quantitative values of three calibration curves are given in Table 6.3.

Consequently, different calibration curves using the FEM simulation (i.e., Calibration Nos. 1, 2 and 3) are compared in Figure 6.7b with those in the literature from Johnson and Cook [63] and Choung et al. [64]. Further, the FE results based on three different calibration curves are compared in Figure 6.8 with the experimental results of the specimen A2.4-12-00 to attain the best calibration curve for the fracture model. In particular, by means of shear domination in this typical configuration, the calibration procedure is conducted based on the range of average stress triaxiality less than 0.4, as referred to in Figure 6.7a. Therefore, it is recommended to change the slope (lower fracture strain curve produces lower ductility) of the calibration curves in this range. In addition, for the remaining range (i.e., the average stress triaxiality larger than 0.4), the fracture due to void formation in the tension zone has successfully captured the behaviour of the actual test across the full load-displacement range where tension fracture then shear fracture occurs. As a result, Calibration No. 1 curve provides the best results against the remaining cases. Therefore, Calibration No. 1 curve has been chosen as the proposed failure criterion for G450 sheet steels and is also employed throughout for all models as investigated in this thesis.

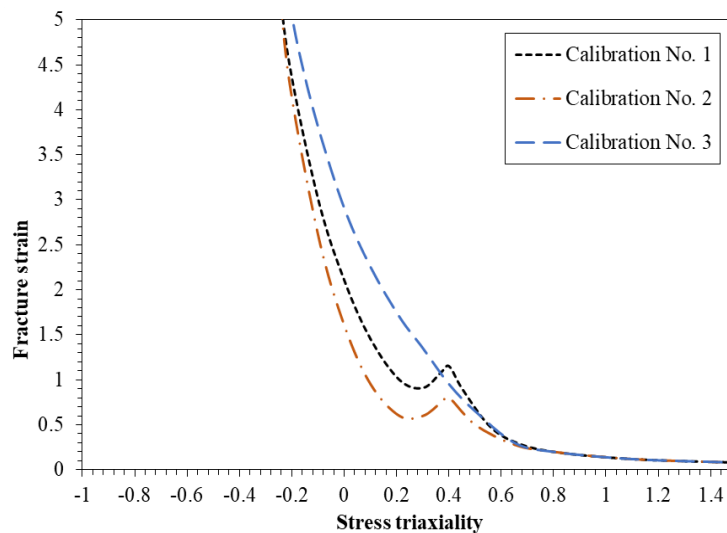


Figure 6.6: Dependence of the equivalent strain to fracture on the stress triaxiality

Table 6.3: Quantitate values of different calibration curves for the ductile fracture model for high strength cold-formed steel

Calibration No. 1 curve		Calibration No. 2 curve		Calibration No. 3 curve	
Fracture strain	Stress triaxiality	Fracture strain	Stress triaxiality	Fracture strain	Stress triaxiality
6.5	-0.265	6.5	-0.265	6.5	-0.265
5.5	-0.25	5.5	-0.25	6	-0.25
4.5	-0.21	4.3	-0.21	5.2	-0.21
3	-0.1	2.6	-0.1	4.6	-0.17
2.1	0	1.6	0	3.8	-0.1
1.45	0.1	0.95	0.1	2.9	0
1	0.21	0.6	0.21	2.25	0.1
0.91	0.3	0.6	0.3	1.7	0.21
1.085	0.37	0.75	0.37	1.35	0.3
1.15	0.4	0.8	0.4	0.95	0.4
0.9	0.45	0.5	0.5	0.65	0.5
0.5	0.55	0.28	0.65	0.3	0.65
0.31	0.65	0.2	0.8	0.2	0.8
0.2	0.8	0.12	1.1	0.12	1.1
0.12	1.1	0.08	1.5	0.08	1.5
0.08	1.5	6.5	-0.265	6.5	-0.265

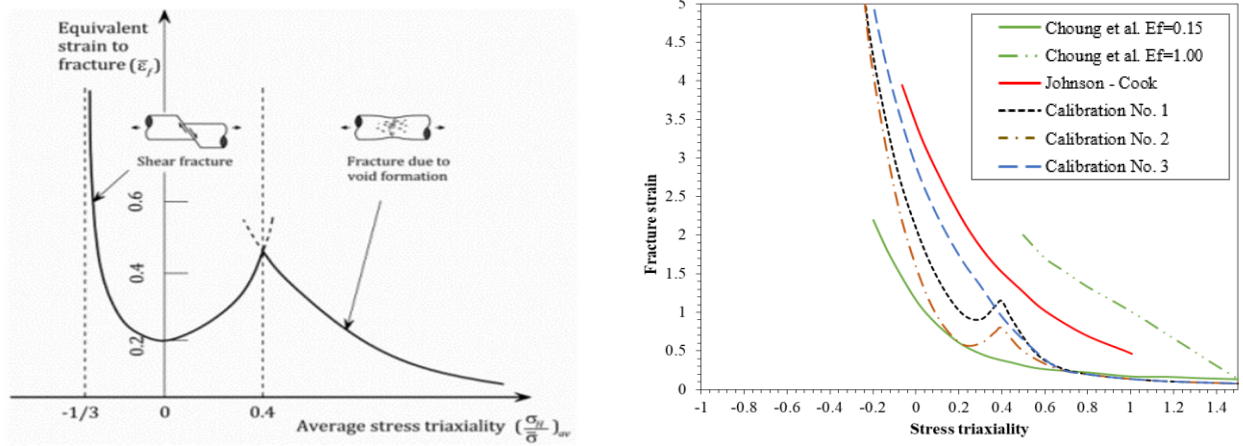


Figure 6.7: (a) Influence of stress triaxiality on the equivalent strain to fracture (Suárez et al. [67]) and (b) various fracture strain formulas

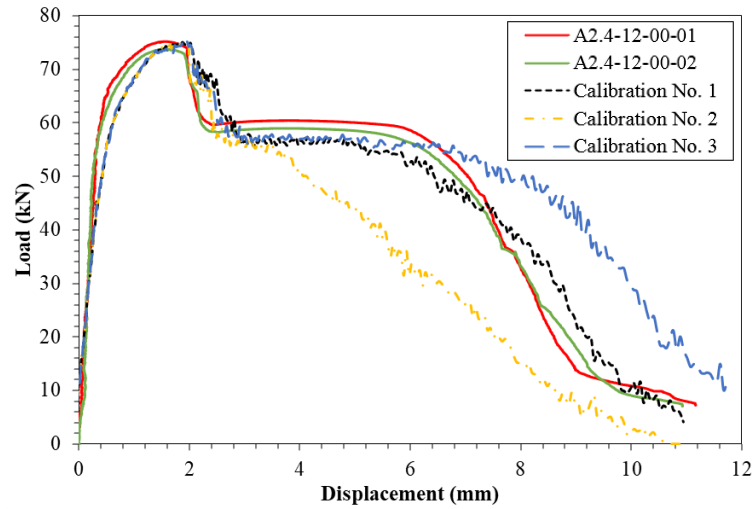


Figure 6.8: Comparison of the FE results based on three different calibration curves with the experiments of the 2.4mm cold-formed steel staggered hole bolted connections using M12-Bolts with the angle $\alpha = 0^\circ$

6.3.1.2 Finite element results

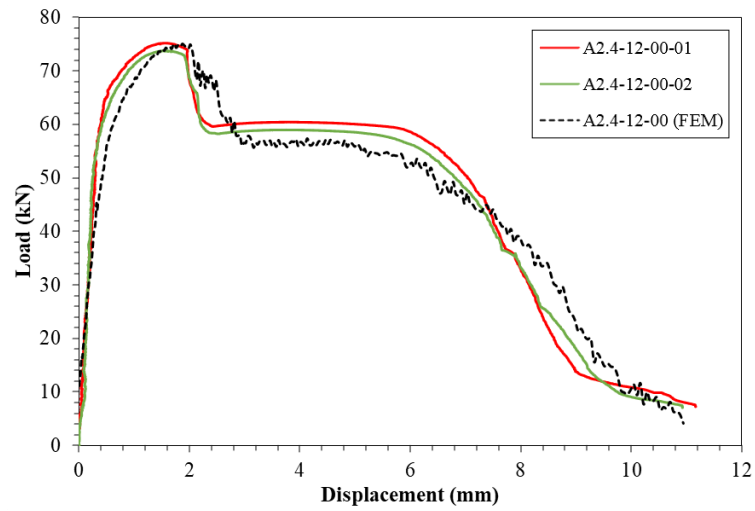
The FE results of the staggered hole bolted connections are given in Table 6.4. It is obvious that all the FE results are approximately similar with the test results through the average ratios of FE results divided by test results and their corresponding coefficient of variation (i.e., Mean = 1.02 and C.o.V = 1.10%), although almost FE results are slightly higher than the test results due to the imperfections in the actual tests (i.e., homogeneous material of sheet steels, symmetric conditions of test set up, etc.). Figure 6.9-Figure 6.12 illustrate the relationship between the displacement and the shear strength of the staggered hole bolted connections using the FEM against the experiments in order to further confirm the validation of the current simulation. It is obvious that the calibrated FE results provide a good agreement with the test results. For instance, in Figure 6.9a, the shear strength of the connections based on the FE curve increases monotonically with the test curves before dropping down to the same value as the experimental results. The long plateau at the end is dominated by shear behaviour before failing totally. In terms of the angle $\alpha = 27^\circ$ and the angle $\alpha = 45^\circ$ results, the failure states in the experiments involve the tension failure, then fracture in the inclined shear zone and finally shear failure, as can be seen in Figure 6.9b & Figure 6.9c. Similar behaviour is observed in the FE modelling throughout. In addition, the slopes of the load displacement curves based on the FE modelling over the whole range follow closely with those of the experimental results. However, there is a slight difference in the ductility between the experimental results and FEM results in most cases that come from the imperfections in the actual tests.

The failure process of the specimen A2.4-12-45 using the DIC technique is compared with the FE results, as demonstrated in Figure 6.13. In the elastic region, the strain occurs locally in

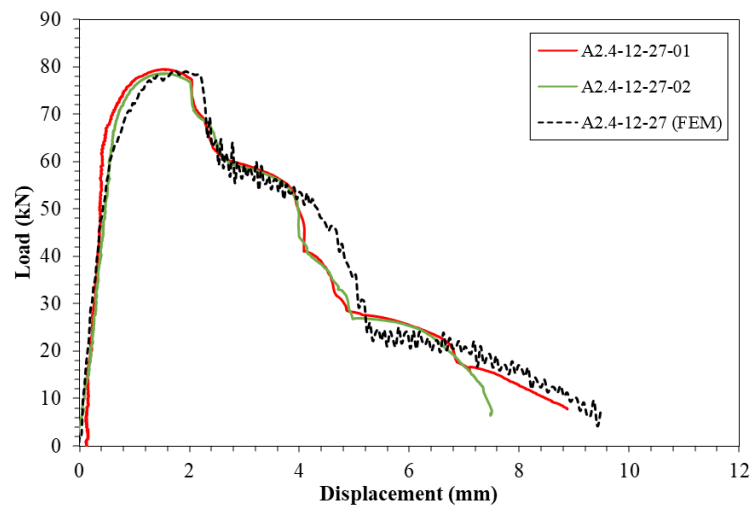
front of the four bolts followed by further extension in the yield phase. After that, the strain in the tension zone and the inclined shear zone speeds up noticeably before fracture occurs between the two bolts. Consequently, the failure propagation in the inclined shear zone occurs almost simultaneously in comparison with the tension region. Finally, a substantial extension is observed by the shear failure in the outside shear region. It can be seen from Figure 6.13 that the fracture propagation based on FEM is similar to that observed in the experiments.

Table 6.4: Comparison of the FE results with the experimental results of staggered holes bolted connections of cold-formed sheets of thickness of 1.9mm and 2.4mm

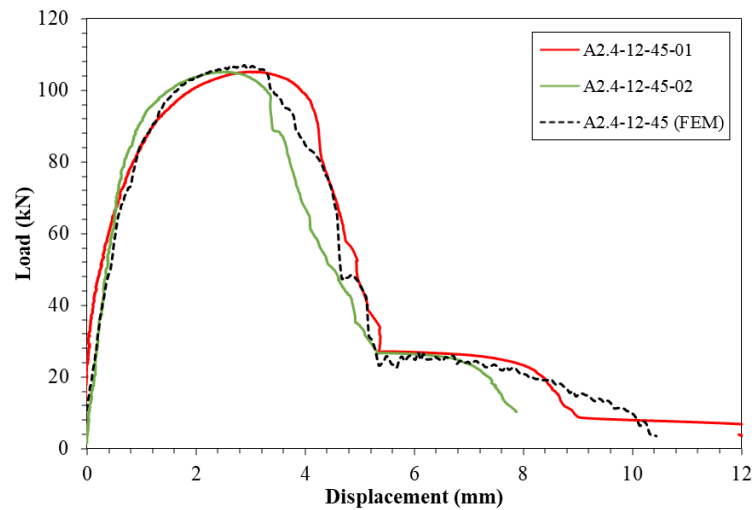
Specimen	Nominal thickness t (mm)	Test results P_1^T (kN)	Test results P_2^T (kN)	Average test results P^T (kN)	FE results P^{FE} (kN)	$\frac{P^{FE}}{P^T}$
A2.4-12-00	2.40	75.19	73.75	74.47	75.06	1.01
A2.4-12-27	2.40	79.46	78.59	79.025	79.12	1.00
A2.4-12-45	2.40	105.20	103.30	104.25	107.06	1.03
A2.4-08-00	2.40	46.85	46.54	46.695	47.16	1.01
A2.4-08-29	2.40	49.44	48.95	49.195	50.18	1.02
A2.4-08-45	2.40	64.81	65.46	65.135	65.76	1.01
A1.9-12-00	1.90	58.94	58.56	58.75	59.46	1.01
A1.9-12-27	1.90	62.12	63.03	62.575	63.37	1.01
A1.9-12-45	1.90	83.58	81.51	82.545	84.58	1.02
A1.9-08-00	1.90	36.21	35.81	36.01	37.52	1.04
A1.9-08-29	1.90	39.05	38.83	38.94	39.89	1.02
A1.9-08-45	1.90	51.33	50.75	51.04	52.35	1.03
					Mean	1.02
					C.o.V (%)	1.10



(a)

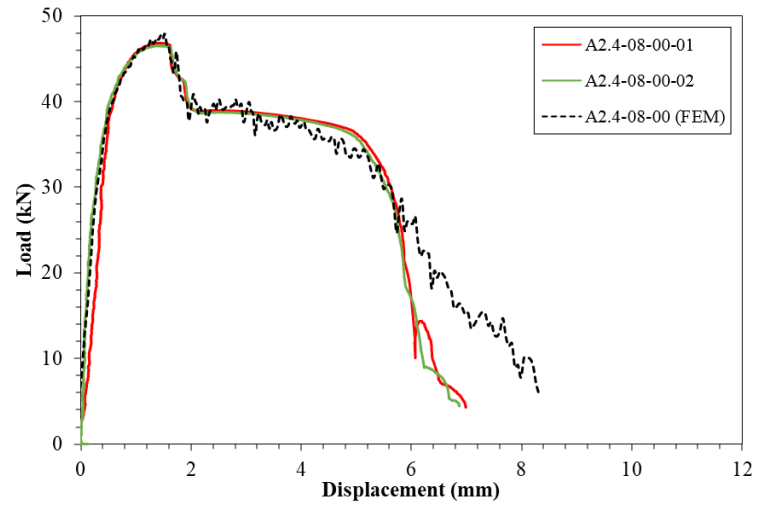


(b)

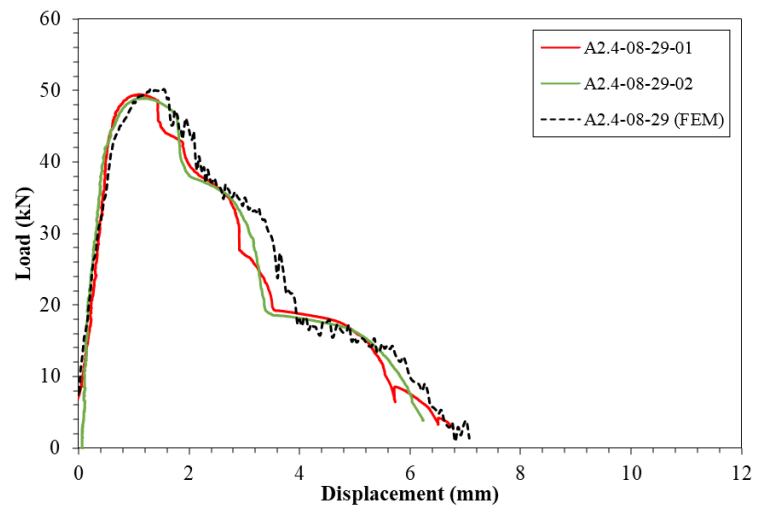


(c)

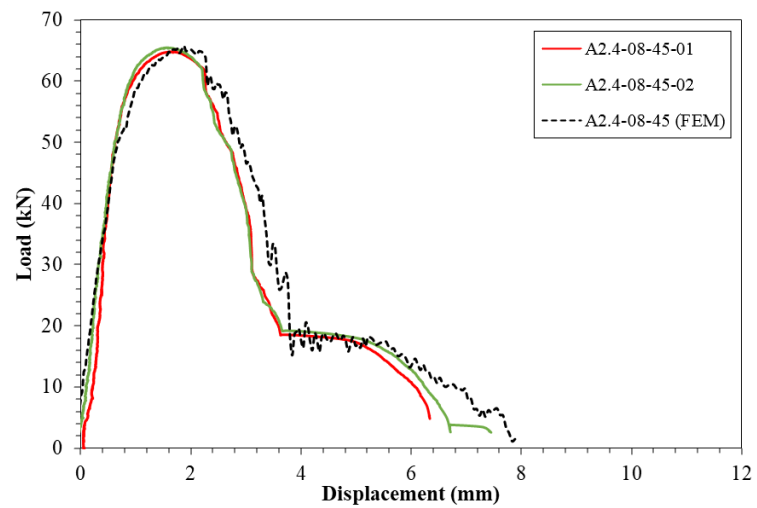
Figure 6.9: Comparison of the FE results with the experiments of the 2.4mm cold-formed steel staggered hole bolted connections using M12-Bolts (a) with the angle $\alpha = 0^\circ$, (b) with the angle $\alpha = 27^\circ$ and (c) with the angle $\alpha = 45^\circ$



(a)

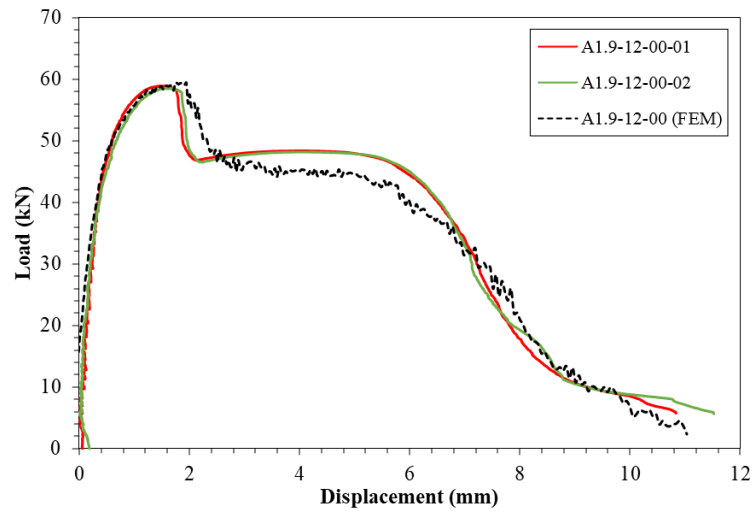


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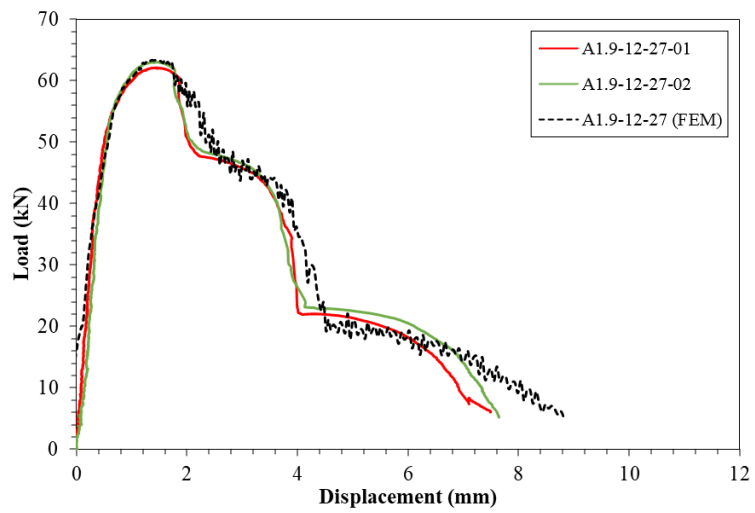


(c)

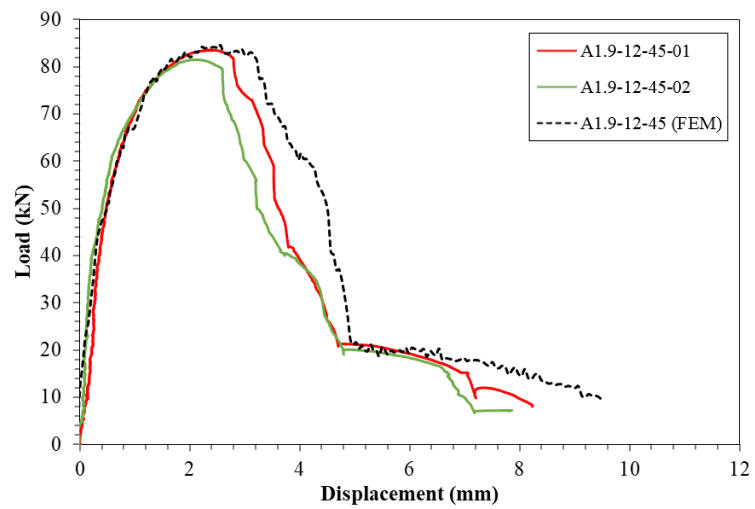
Figure 6.10: Comparison of the FE results with the experiments of the 2.4mm cold-formed steel staggered hole bolted connections using M8-Bolts (a) with the angle $\alpha = 0^\circ$, (b) with the angle $\alpha = 29^\circ$ and (c) with the angle $\alpha = 45^\circ$



(a)

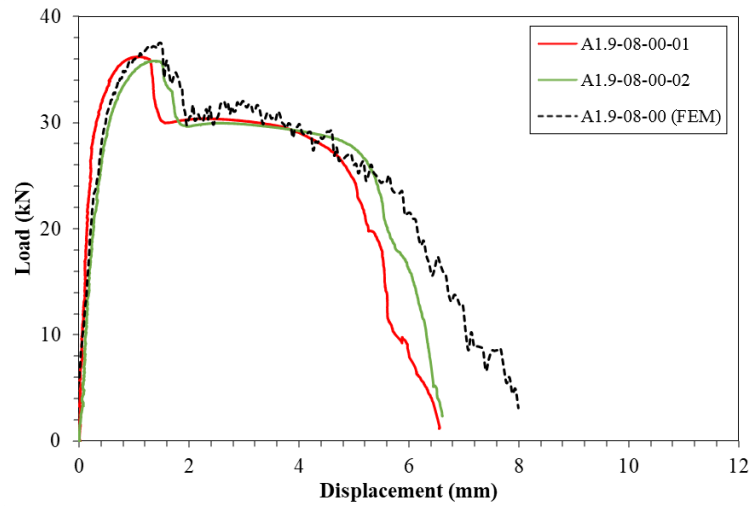


(b)

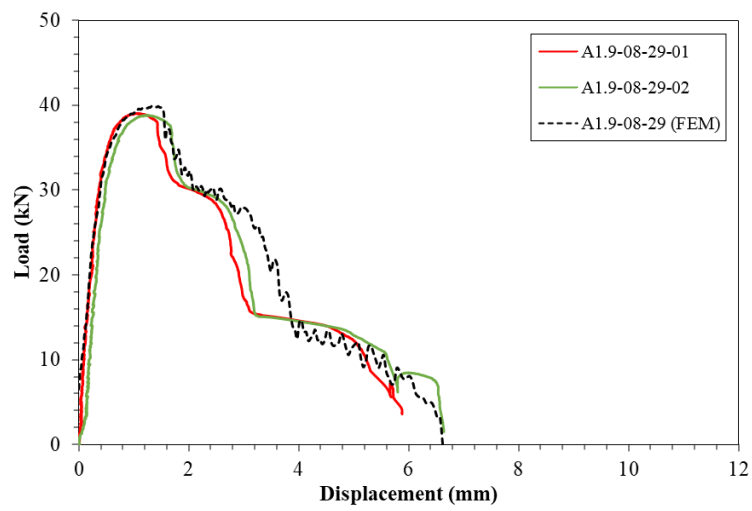


(c)

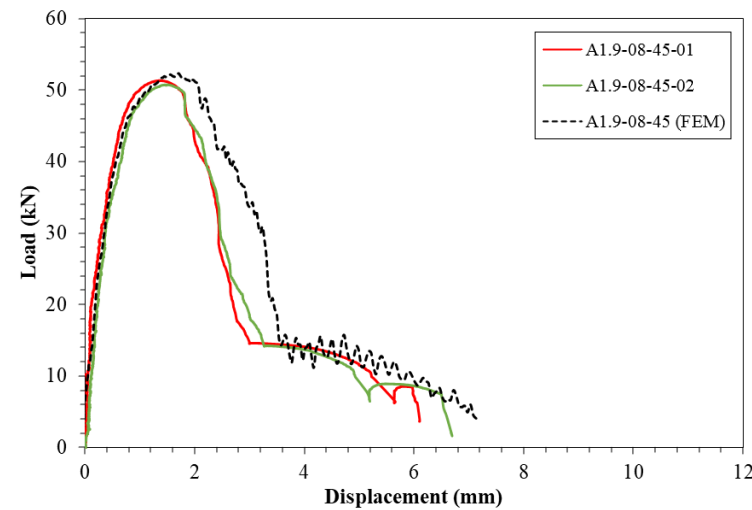
Figure 6.11: Comparison of the FE results with the experiments of the 1.9mm cold-formed steel staggered hole bolted connections using M12-Bolts (a) with the angle $\alpha = 0^\circ$, (b) with the angle $\alpha = 27^\circ$ and (c) with the angle $\alpha = 45^\circ$



(a)



(b)



(c)

Figure 6.12: Comparison of the FE results with the experiments of the 1.9mm cold-formed steel staggered hole bolted connections using M8-Bolts (a) with the angle $\alpha = 0^\circ$, (b) with the angle $\alpha = 29^\circ$ and (c) with the angle $\alpha = 45^\circ$

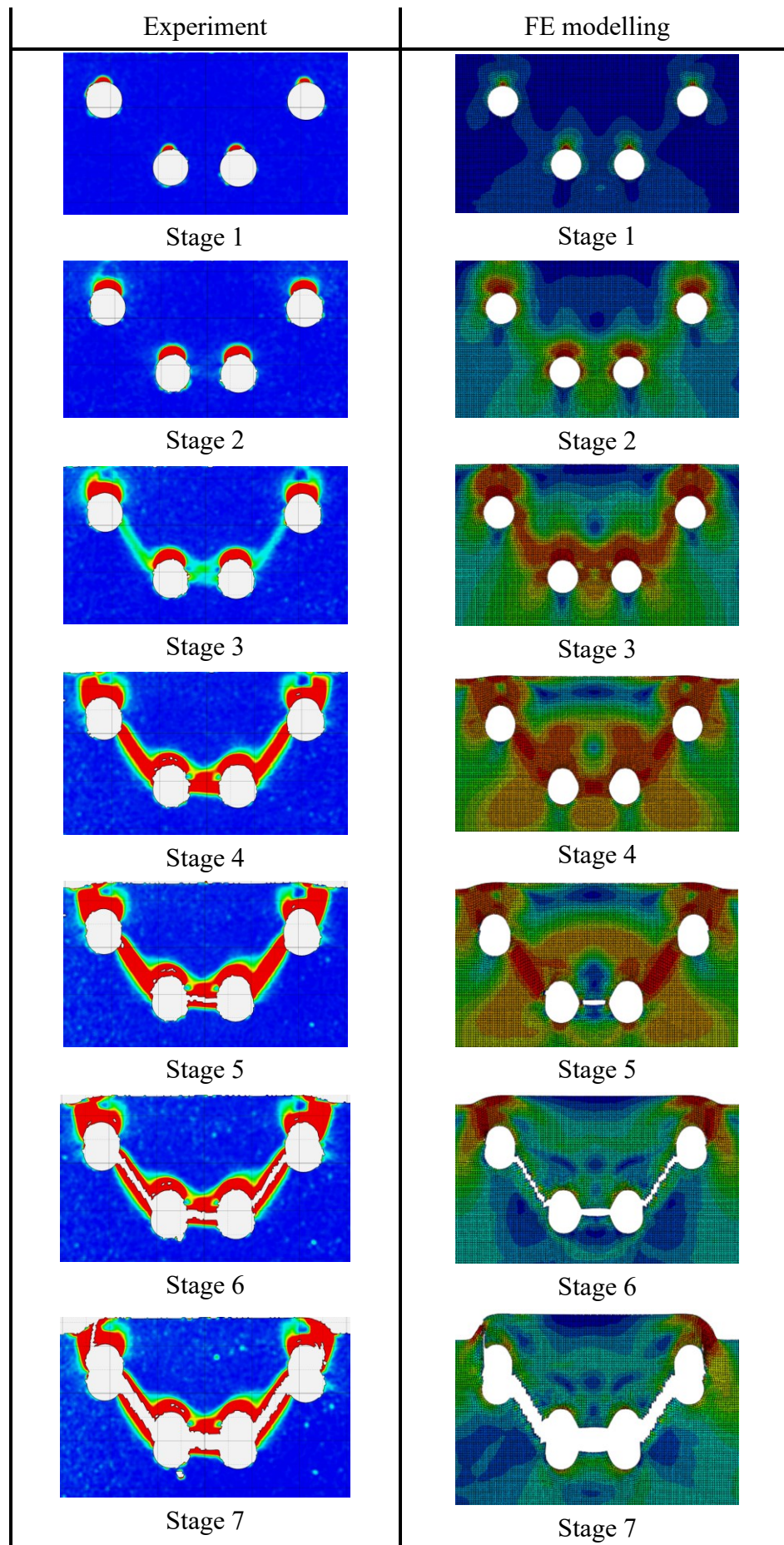
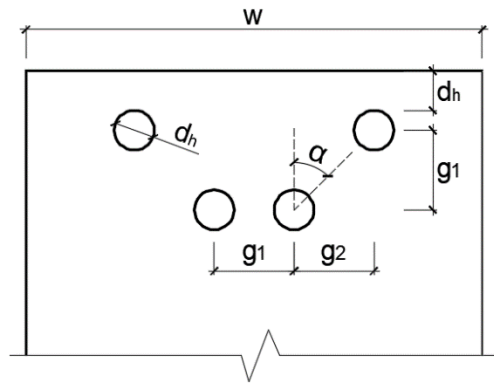


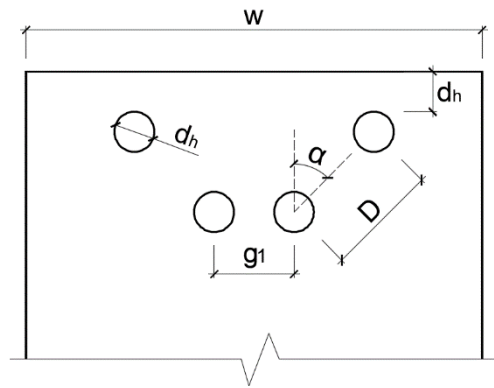
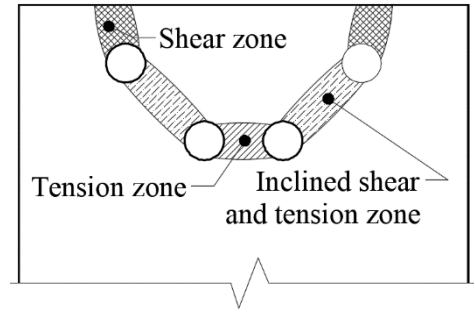
Figure 6.13: Comparison of failure process of the specimen A2.4-12-45 from FE modelling and from experiments

6.3.1.3 Parametric study

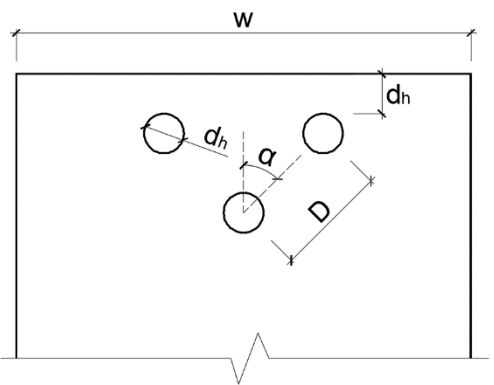
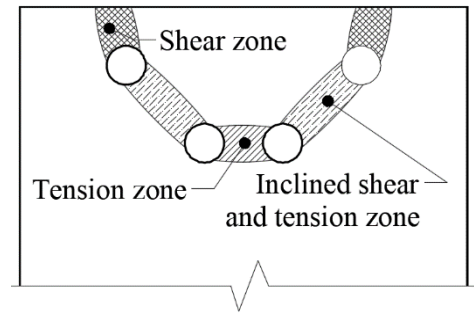
Four different types of configurations as can be seen in detail in Figure 6.14 and Table 6.5-Table 6.8 of staggered hole bolted connections are modelled to investigate the effect of the inclined shear and tension zones on the block shear strength and behaviour of the connections. In particular, the Configuration Type A in Figure 6.14a is similar to the previous tests except that more cases of the angle α are added. The length between the two stagger holes (i.e., D in Figure 6.14b) is unchanged in Configuration Type B to investigate only the influence of the angle α on the block shear strength of the connections. Although both Configurations Type A and B have a similar configuration, the inclined shear and tension area in the Configuration Type B is unchanged (D is constant). Therefore, the block shear strength of the connection Configuration Type B only depends on the change of the angle α . In Configuration Type C shown in Figure 6.14c, only three holes of the staggered hole bolted connections are investigated which may be more practical in reality in comparison with the four hole connections. In addition, this special configuration is studied to focus on the effect of the inclined shear and tension zone on the block shear strength and behaviour of the connections without the presence of the pure tension zone. A similar configuration is analysed in Configuration Type D shown in Figure 6.14d using five holes in order to increase the influence of the staggered paths on the connections.



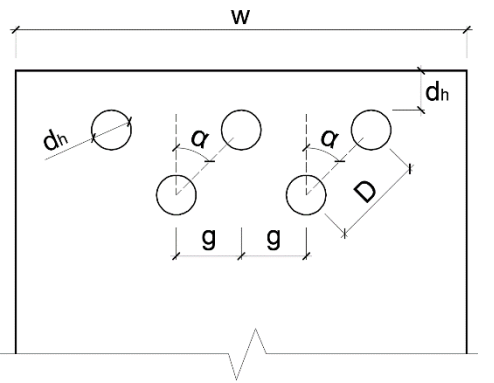
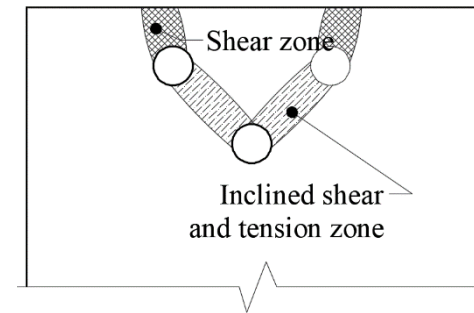
(a)



(b)



(c)



(d)

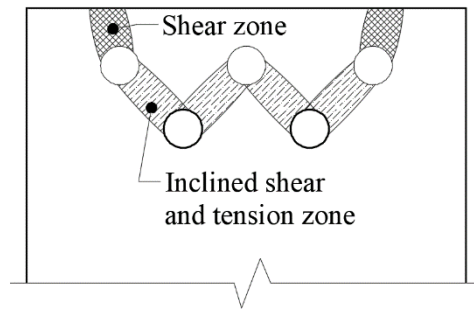


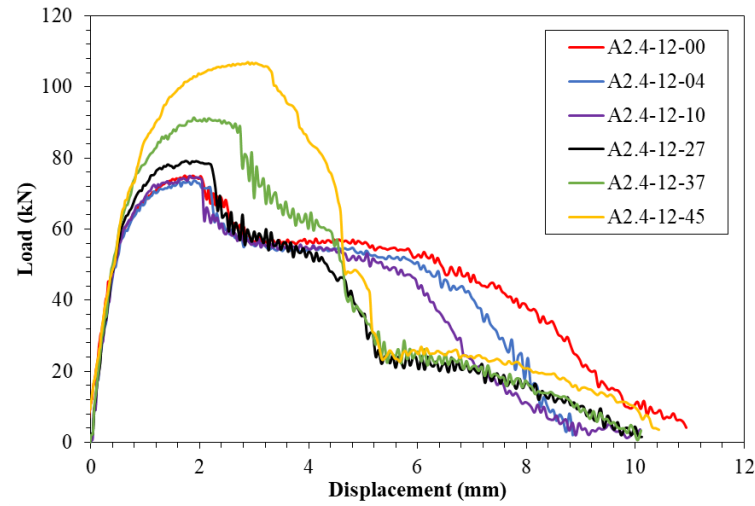
Figure 6.14: Configuration and zones of strain extension of staggered hole bolted connections (a) Configuration Type A, (b) Configuration Type B, (c) Configuration Type C and (d) Configuration Type D.

Table 6.5 summaries the FE results of the block shear strength of the staggered holes bolted connections Configuration Type A. It is pointed out that the ultimate block shear strength of the connections is influenced significantly by the inclined shear via the change of the angle α . In particular, in the case of the 2.4 mm cold-formed steel sheets with 14mm hole size, the block shear strength of the connections increases slightly from $\alpha = 0^\circ$ to $\alpha = 27^\circ$ while a large change occurs when the angle α reaches to 45° . The block shear strength goes up by 35% between 27° and 45° whereas only 5% improvement occurs from 0° to 27° . Similar behaviour is observed in the remaining cases of staggered hole bolted connections Configuration Type A. Overall, the block shear strength of 2.4 mm cold-formed steel staggered hole bolted connections using M8-Bolts is nearly lower 37% than that of 2.4 mm cold-formed steel staggered hole bolted connections using M12-Bolts. The influence of the thickness of the cold-formed steel sheets on the block shear strength is demonstrated in Table 6.5. The ultimate shear strengths of the staggered hole bolted connections using 1.9 mm high strength steel sheets decrease approximately by 21% in comparison with that of the staggered hole bolted connections using 2.4 mm high strength steel sheets although the effect of the angle α on the block shear strength of the connections is similar to the first case (i.e., $t = 2.4\text{mm}$).

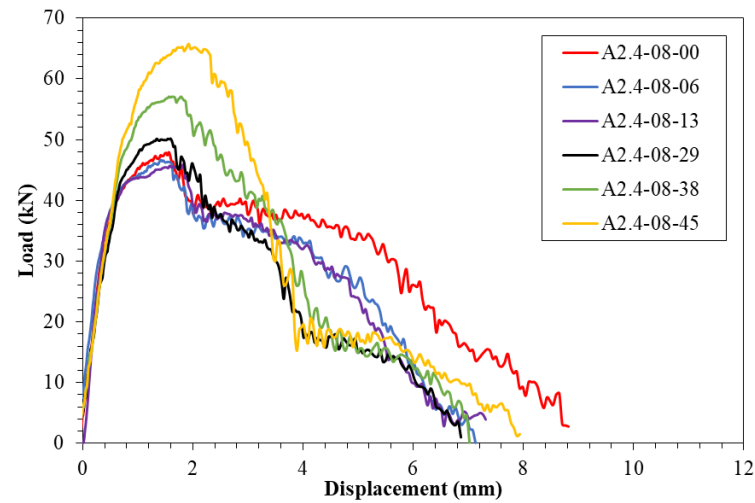
The relationship between the displacement and the block shear strength of the staggered hole bolted connections is demonstrated in Figure 6.15. Details of the full set of load-displacement curves are given in Appendix E. It is clear that the calibrated FE modelling provides consistent results for all cases. In particular in Figure 6.15a, the behaviour of the connections in the case of quite small α (i.e., $\alpha = 4^\circ$) is similar to the case of traditional rectangular block shear connections. After failing in the tension zone between the two lower bolts, the connection experiences a substantial extension of the strain in the shear zones resulting in an improvement of the ductility of the connection. In addition, the behaviour of the three remaining curves (i.e., $\alpha = 27^\circ, 37^\circ, 45^\circ$) is nearly the same where the block shear strength drops significantly in the inclined shear and tension zones with the high deformation (i.e., 5mm displacement) after failing in the tension zone. It demonstrates that the shear failure dominates in the inclined shear and tension zones when the angle $\alpha \leq 45^\circ$. Further, similar behaviour is observed in the remaining cases (i.e., Figure 6.15b) as proof of shear rupture domination in the inclined shear and tension zones when the angle $\alpha \leq 45^\circ$.

Table 6.5: FE results of staggered holes bolted connections Configuration Type A of cold-formed sheets of thickness of 1.9mm and 2.4mm

Specimen	Nominal thickness t (mm)	Nominal width w (mm)	Nominal hole size d_h (mm)	Nominal bolt size d_f (mm)	Gauge g_1 (mm)	Gauge g_2 (mm)	Angle α ($^\circ$)	FE results P^{FE} (kN)
A2.4-12-00	2.40	160	14	12	28	0	0	75.06
A2.4-12-04	2.40	160	14	12	28	2	4	74.16
A2.4-12-10	2.40	160	14	12	28	5	10	75.09
A2.4-12-27	2.40	160	14	12	28	14	27	79.12
A2.4-12-37	2.40	160	14	12	28	21	37	91.35
A2.4-12-45	2.40	160	14	12	28	28	45	107.06
A2.4-08-00	2.40	160	10	08	18	0	0	47.16
A2.4-08-06	2.40	160	10	08	18	2	6	46.71
A2.4-08-13	2.40	160	10	08	18	4	13	45.91
A2.4-08-29	2.40	160	10	08	18	10	29	50.18
A2.4-08-38	2.40	160	10	08	18	14	38	57.09
A2.4-08-45	2.40	160	10	08	18	18	45	65.76
A1.9-12-00	1.90	160	14	12	28	0	0	59.46
A1.9-12-04	1.90	160	14	12	28	2	4	58.74
A1.9-12-10	1.90	160	14	12	28	5	10	59.01
A1.9-12-27	1.90	160	14	12	28	14	27	63.37
A1.9-12-37	1.90	160	14	12	28	21	37	72.11
A1.9-12-45	1.90	160	14	12	28	28	45	84.58
A1.9-08-00	1.90	160	10	08	18	0	0	37.52
A1.9-08-06	1.90	160	10	08	18	2	6	36.22
A1.9-08-13	1.90	160	10	08	18	4	13	35.85
A1.9-08-29	1.90	160	10	08	18	10	29	39.89
A1.9-08-38	1.90	160	10	08	18	14	38	45.57
A1.9-08-45	1.90	160	10	08	18	18	45	52.35



(a)



(b)

Figure 6.15: FE results of 2.4mm cold-formed steel staggered hole bolted connections Configuration Type A using (a) M12-Bolts and (b) M8-Bolts.

By following the same FE modelling procedure as employed in the first type of configuration (i.e., staggered hole bolted connections Configuration Type A), the behaviour and block shear strength of the staggered hole bolted connections Configuration Type B is further investigated to determine the effect of the angle α on the connections without changing the inclined length (D) between the staggered holes. The FE results of the staggered holes bolted connections Configuration Type B are provided in Table 6.6. It is noted that the effect of the angle α on the ultimate shear strength is quite similar as the first type of configuration. In particular, the block shear strength of the connections improves steadily from $\alpha = 0^\circ$ to $\alpha = 30^\circ$ whereas it increases significantly by approximately 11% when the angle α increases from 30° to 60° before increasing monotonically by 5% in the case of pure tension failure (i.e., $\alpha = 90^\circ$).

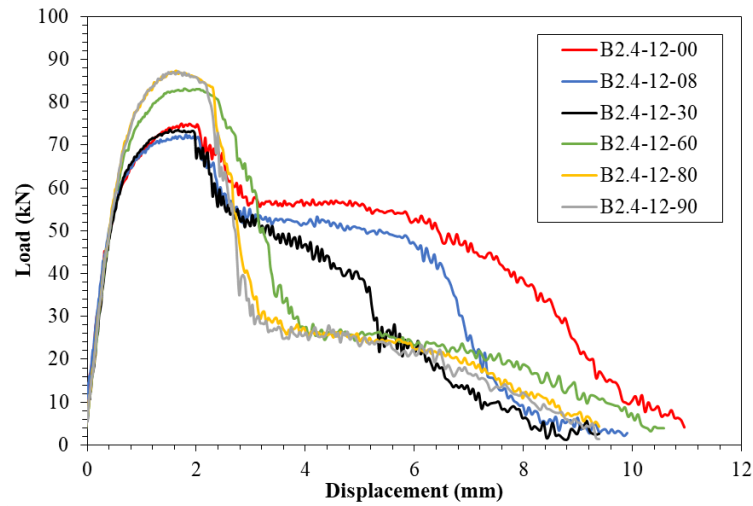
Similar behaviours are observed for the remaining cases which confirms the consistency of the FE modelling procedure.

Further, Figure 6.16 illustrates the relationship between the displacement and the block shear strength of the staggered hole bolted connections Configuration Type B. In particular, the block shear strength strongly correlates to the angle α . For instance, in Figure 6.16a, the load-displacement curves in the cases of $\alpha = 0^\circ, 8^\circ, 30^\circ$ have the same trend in which after failing in the tension zone between the two lower bolts, the connections experience a substantial extension of the strain in the shear zones ($\alpha = 0^\circ$) or inclined shear and tension zones ($\alpha = 8^\circ, 30^\circ$) resulting in an improvement of the ductility of the connections. This behaviour confirms the shear rupture domination in the inclined shear and tension zones when the angle $\alpha \leq 30^\circ$. However, in the cases of $\alpha = 60^\circ, 80^\circ, 90^\circ$, the block shear strength drops sharply in the inclined shear and tension zones ($\alpha = 60^\circ, 80^\circ$) or pure tension zones ($\alpha = 90^\circ$) after failing in the tension zone between the two lower bolts. In addition, the connections in these cases are less ductile than the cases of the small angle α (i.e., $\alpha \leq 30^\circ$) which confirms the tension rupture domination in the inclined shear and tension zones when the angle $\alpha \geq 60^\circ$.

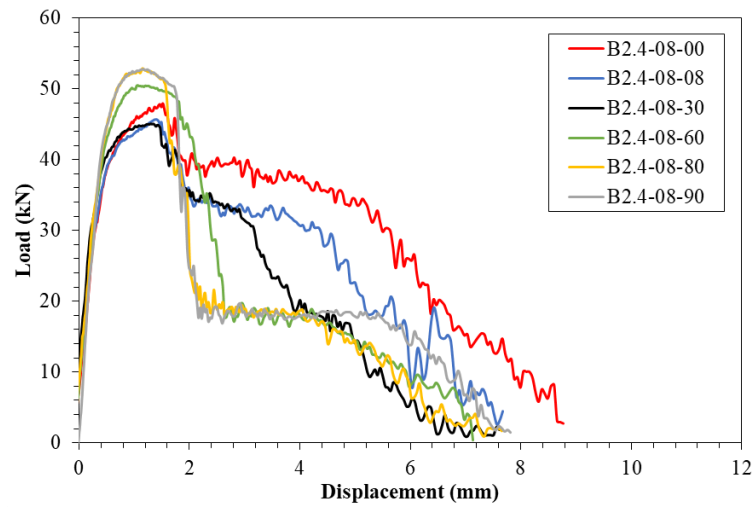
By comparison, it is apparent from Figure 6.16b that the behaviour of the remaining load-displacement curves has the same trend as that observed in Figure 6.16a which further demonstrates the shear failure domination in the inclined shear and tension zones when the angle $\alpha \leq 30^\circ$ and tension failure domination in the inclined shear and tension zones when the angle $\alpha \geq 60^\circ$. Details of the full set of load-displacement curves are given in Appendix E.

Table 6.6: FE results of staggered holes bolted connections Configuration Type B of cold-formed sheets of thickness of 1.9mm and 2.4mm

Specimen	Nominal thickness t (mm)	Nominal width w (mm)	Nominal hole size d_h (mm)	Gauge g_1 (mm)	Inclined D (mm)	Angle α ($^\circ$)	FE results P^{FE} (kN)
B2.4-12-00	2.40	160	14	28	28	0	75.06
B2.4-12-08	2.40	160	14	28	28	8	72.53
B2.4-12-30	2.40	160	14	28	28	30	73.68
B2.4-12-60	2.40	160	14	28	28	60	83.17
B2.4-12-80	2.40	160	14	28	28	80	87.35
B2.4-12-90	2.40	160	14	28	28	90	87.24
B2.4-08-00	2.40	160	10	18	18	0	47.16
B2.4-08-08	2.40	160	10	18	18	8	45.72
B2.4-08-30	2.40	160	10	18	18	30	45.07
B2.4-08-60	2.40	160	10	18	18	60	50.51
B2.4-08-80	2.40	160	10	18	18	80	52.85
B2.4-08-90	2.40	160	10	18	18	90	52.84
B1.9-12-00	1.90	160	14	28	28	0	59.46
B1.9-12-08	1.90	160	14	28	28	8	57.61
B1.9-12-30	1.90	160	14	28	28	30	58.07
B1.9-12-60	1.90	160	14	28	28	60	66.09
B1.9-12-80	1.90	160	14	28	28	80	69.40
B1.9-12-90	1.90	160	14	28	28	90	69.18
B1.9-08-00	1.90	160	10	18	18	0	37.52
B1.9-08-08	1.90	160	10	18	18	8	35.88
B1.9-08-30	1.90	160	10	18	18	30	35.50
B1.9-08-60	1.90	160	10	18	18	60	39.96
B1.9-08-80	1.90	160	10	18	18	80	41.63
B1.9-08-90	1.90	160	10	18	18	90	41.54



(a)



(b)

Figure 6.16: FE results of 2.4mm cold-formed steel staggered hole bolted connections Configuration Type B using (a) M12-Bolts and (b) M8-Bolts.

The FE results of the staggered holes bolted connections Configuration Type C is demonstrated in Table 6.7. In this configuration, only three hole connections are modelled to study the effect of the inclined shear and tension zone on the connections without the presence of the tension zone. It is apparent that the ultimate block shear strength of the connections increases monotonically by 6-8% corresponding to the increase of each angle α for all cases.

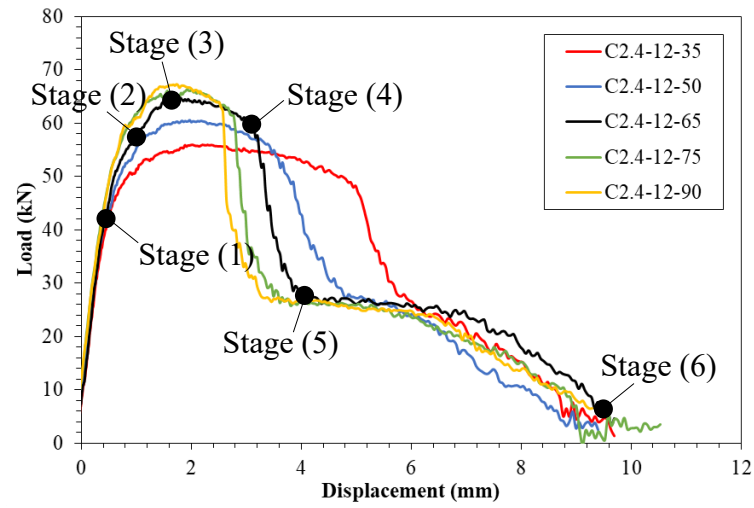
The relationship between the displacement and the block shear strength of the staggered hole bolted connections Configuration Type C is shown in Figure 6.17. In particular in Figure 6.17a, the connections directly fail in the inclined shear and tension zones before experiencing a substantial extension of the strain in the remaining shear zones. In addition, the load-displacement curves change steadily from $\alpha = 35^\circ$ to $\alpha = 90^\circ$ without any discontinuity as observed from the last two configurations (i.e., Configuration Types A and B), which also implies the simultaneous

effect of both shear and tension ruptures on the staggered zones of the staggered hole bolted connections. Similar behaviour is observed for the remaining cases in Figure 6.17b demonstrating the consistency of the FE procedure. Details of the full set of load-displacement curves are given in Appendix E.

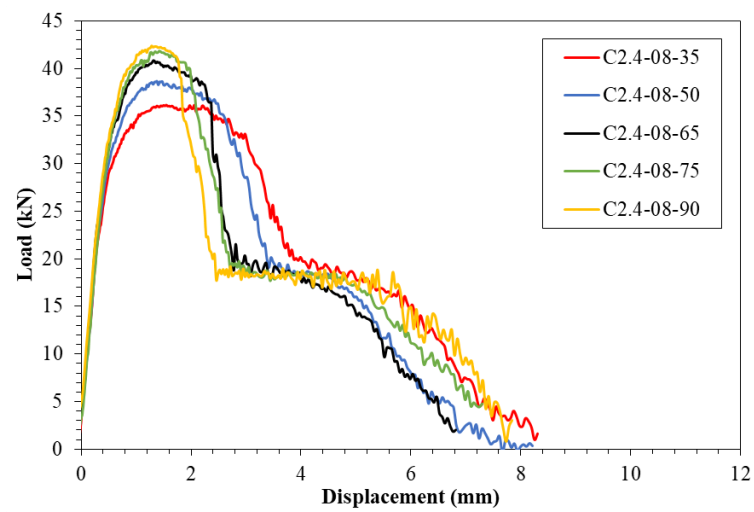
The failure process of the specimen C2.4-12-65 using FE modelling is demonstrated in Figure 6.18. In particular, the strain in the elastic region (e.g., Stage 1) only appears locally in front of the three bolts before extending further in the yield phase (e.g., Stage 2). Consequently, the strain in the inclined shear and tension zone increases significantly (e.g., Stage 3) before fracture appears between the two bolts as a result of critical strain (e.g., Stages 4 & 5), followed by a substantial extension in the shear zone due to the shear failure (e.g., Stage 6).

Table 6.7: FE results of staggered holes bolted connections Configuration Type C of cold-formed sheets of thickness of 1.9mm and 2.4mm

Specimen	Nominal thickness t (mm)	Nominal width w (mm)	Nominal hole size d_h (mm)	Nominal bolt size d_f (mm)	Inclined D (mm)	Angle α ($^\circ$)	FE results P^{FE} (kN)
C2.4-12-35	2.40	160	14	12	28	35	55.90
C2.4-12-50	2.40	160	14	12	28	50	60.63
C2.4-12-65	2.40	160	14	12	28	65	64.75
C2.4-12-75	2.40	160	14	12	28	75	66.42
C2.4-12-90	2.40	160	14	12	28	90	67.29
C2.4-08-35	2.40	160	10	08	18	35	36.17
C2.4-08-50	2.40	160	10	08	18	50	38.64
C2.4-08-65	2.40	160	10	08	18	65	40.83
C2.4-08-75	2.40	160	10	08	18	75	41.84
C2.4-08-90	2.40	160	10	08	18	90	42.41
C1.9-12-35	1.90	160	14	12	28	35	44.11
C1.9-12-50	1.90	160	14	12	28	50	47.78
C1.9-12-65	1.90	160	14	12	28	65	51.19
C1.9-12-75	1.90	160	14	12	28	75	52.49
C1.9-12-90	1.90	160	14	12	28	90	53.28
C1.9-08-35	1.90	160	10	08	18	35	28.65
C1.9-08-50	1.90	160	10	08	18	50	30.47
C1.9-08-65	1.90	160	10	08	18	65	32.17
C1.9-08-75	1.90	160	10	08	18	75	32.93
C1.9-08-90	1.90	160	10	08	18	90	33.55



(a)



(b)

Figure 6.17: FE results of 2.4mm cold-formed steel staggered hole bolted connections Configuration Type C using (a) M12-Bolts and (b) M8-Bolts.

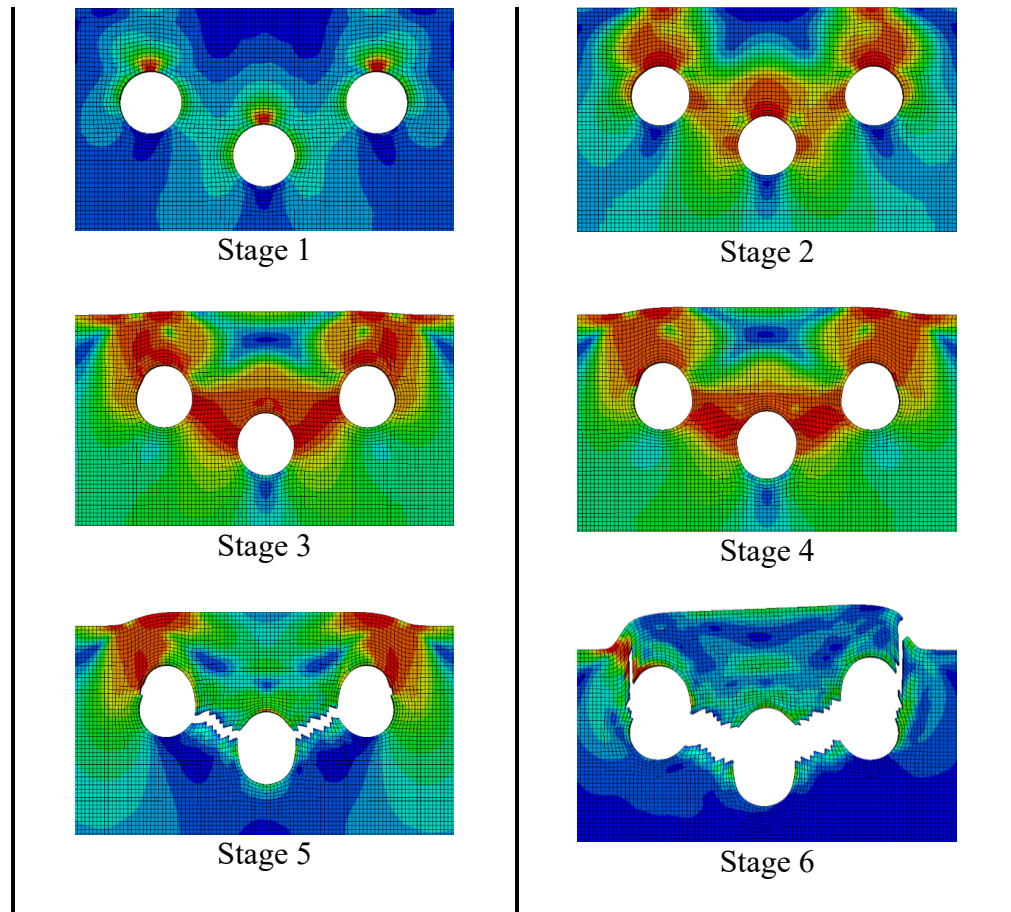


Figure 6.18: Schematic of failure process of the specimen C2.4-12-65 from FE modelling through 6 stages.

Similar FE modelling procedures are employed for the staggered hole bolted connections Configuration Type D using five holes to further investigate the influence of the staggered paths on the connections. The FE results of staggered holes bolted connections using five holes is given in Table 6.8. It is noticeable that the ultimate block shear strength of the connections also raises monotonically by 6-8% corresponding to the increase of each angle α for all cases. Likewise, the ductility of this configuration is exactly the same as the connections Configuration Type C for both cases of M12-bolts and M-8 bolts, accounting for 10mm and 8mm of the displacements, respectively.

Figure 6.19 illustrates the fracture behaviour of both the specimens D2.4-12-30 and D2.4-08-30. The fracture appears in the pure tension zone before extending further in other regions. In particular, the block shear strength of the connections corresponding to this fracture path is smaller than that corresponding to the path shown in Figure 6.14d. It demonstrates another fracture path instead of occurring simultaneously in all the inclined shear and tension zones (Figure 6.14d). Therefore, it is not of interest to investigate these configurations (angle $\alpha \leq 30^\circ$) when calibrating the design equations for staggered holes.

Figure 6.20 shows the relationship between the displacement and the block shear strength of the staggered hole bolted connections Configuration Type D. The behaviour of load-displacement curves has the same trend as the connections Configuration Type C, although more holes are added in the modelling, which also confirms the combined effects of shear and tension ruptures on the staggered zones of the staggered hole bolted connections. Details of the full set of load-displacement curves are given in Appendix E.

Figure 6.21 demonstrates the failure process of the specimen D2.4-12-65 using FE modelling. The fracture propagation is similar to that observed in Figure 6.18. In the elastic range, the strain only localizes in front of the five bolt holes before extending in the hardening range. After the strain in the inclined shear and tension zone increases noticeably when the shear strength increases to the peak, the failure propagation in the inclined shear and tension zone occurs almost simultaneously. Finally, a substantial deformation is observed by the shear failure in the shear region.

Table 6.8: FE results of staggered holes bolted connections Configuration Type D of cold-formed sheets of thickness of 1.9mm and 2.4mm

Specimen	Nominal thickness t (mm)	Nominal width w (mm)	Nominal hole size d_h (mm)	Gauge g (mm)	Inclined D (mm)	Angle α (°)	FE results P^{FE} (kN)
D2.4-12-30	2.40	160	14	12.74	25.6	30	65.48*
D2.4-12-39	2.40	160	14	16.06	25.6	39	75.46
D2.4-12-50	2.40	160	14	19.57	25.6	50	80.90
D2.4-12-65	2.40	160	14	23.17	25.6	65	88.13
D2.4-12-75	2.40	160	14	24.71	25.6	75	91.41
D2.4-12-90	2.40	160	14	25.60	25.6	90	92.17
D2.4-08-30	2.40	160	10	8.20	16.4	30	40.60*
D2.4-08-39	2.40	160	10	10.32	16.4	39	46.90
D2.4-08-50	2.40	160	10	12.57	16.4	50	50.03
D2.4-08-65	2.40	160	10	14.87	16.4	65	52.94
D2.4-08-75	2.40	160	10	15.84	16.4	75	54.82
D2.4-08-90	2.40	160	10	16.40	16.4	90	55.09
D1.9-12-39	1.90	160	14	16.06	25.6	39	59.75
D1.9-12-50	1.90	160	14	19.57	25.6	50	64.08
D1.9-12-65	1.90	160	14	23.17	25.6	65	69.30
D1.9-12-75	1.90	160	14	24.71	25.6	75	71.96
D1.9-12-90	1.90	160	14	25.60	25.6	90	72.85
D1.9-08-39	1.90	160	10	10.32	16.4	39	36.98
D1.9-08-50	1.90	160	10	12.57	16.4	50	39.34
D1.9-08-65	1.90	160	10	14.87	16.4	65	41.82
D1.9-08-75	1.90	160	10	15.84	16.4	75	43.27
D1.9-08-90	1.90	160	10	16.40	16.4	90	43.56

Note: *No interest to investigate because of failure mode.

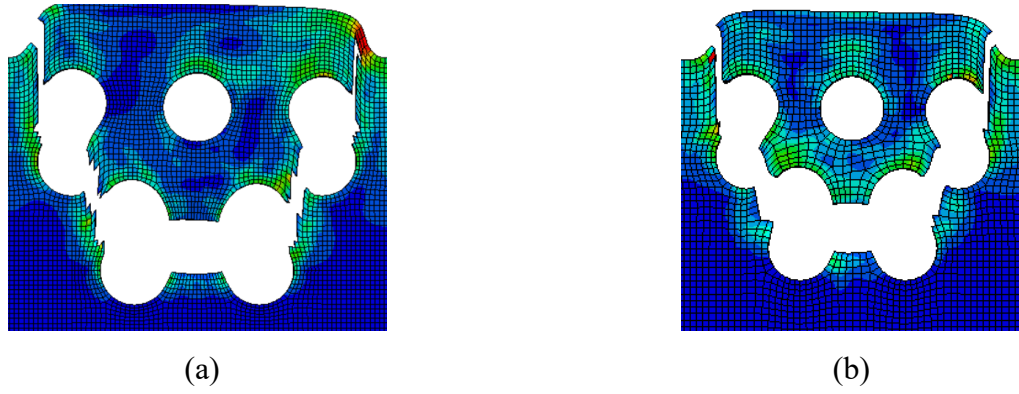


Figure 6.19: Schematic of (a) the specimen D2.4-12-30 and (b) the specimen D2.4-08-30 after failing.

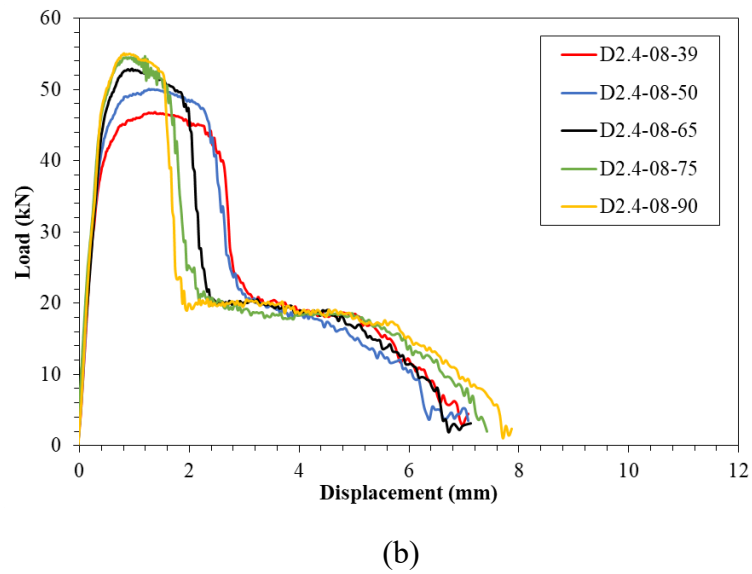
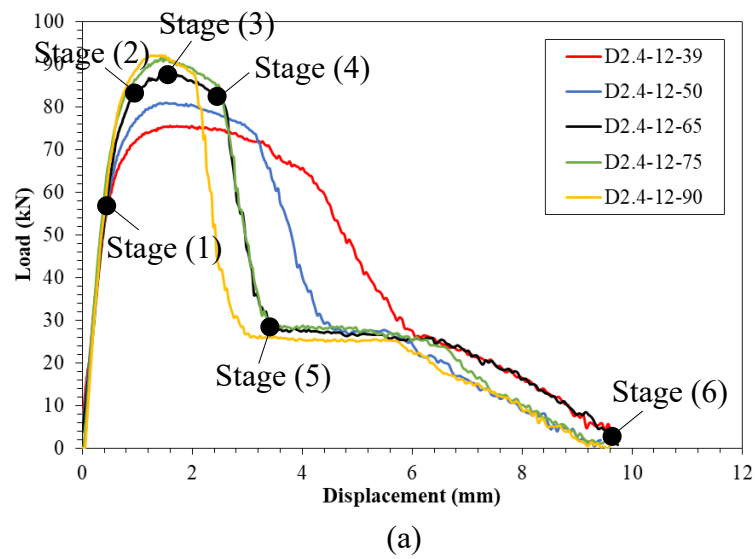


Figure 6.20: FE results of 2.4mm cold-formed steel staggered hole bolted connections Configuration Type D using (a) M12-Bolts and (b) M8-Bolts.

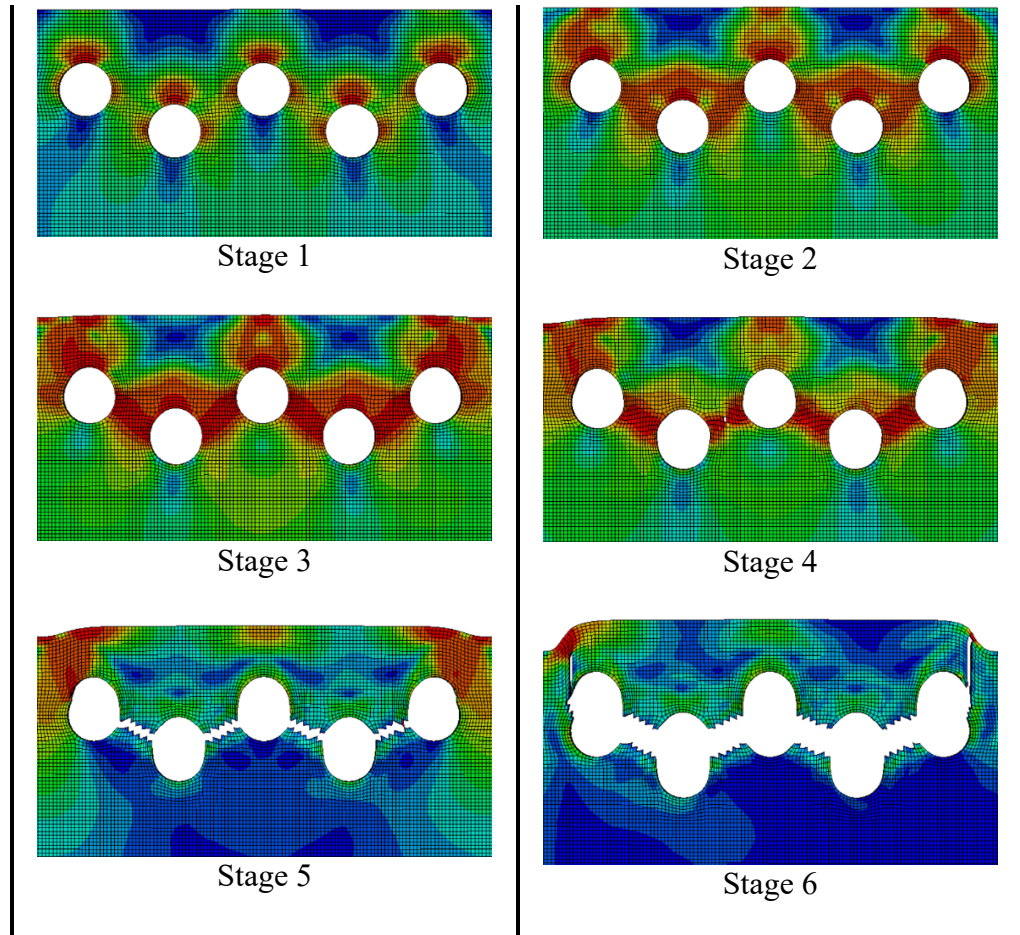


Figure 6.21: Schematic of failure process of the specimen D2.4-12-65 from FE modelling through 6 stages.

Based on the large data set from the FE results, the accuracy of the proposed design Equation (3.2) is further compared with the design standards and specifications from AISI S100:2016 (R2020) w/S3-22 [1] and AS/NZS 4600:2018 [2], as can be seen in Table 6.9. In particular, the ratio of FE results divided by the predicted values ranges from 0.97 to 1.06 while that by AISI varies between 1.00 and 1.29. Again, the proposed formula is more accurate to obtain the nominal block shear strength in the cases of small angle α compared with the AISI due to the approximation of the inclined paths as tension paths in AISI design formulas (i.e., A2.4-12-27, A2.4-08-29, A1.9-12-27 and A1.9-08-29). In addition, the accuracy of the proposed design formula is demonstrated through the average ratios of FE results divided by predicted values and their corresponding coefficient of variation (i.e., Mean = 1.01 and C.o.V = 1.97%) while the AISI predictions are more conservative and inaccurate in obtaining the nominal block shear strength in comparison with the proposed formula (i.e., Mean = 1.08 and C.o.V = 5.78%).

Table 6.9: Comparison of the proposed design Equation (3.2) with the calibrated FE results and the cold-formed steel design standards AISI S100:2016 (R2020) w/S3-22 [1] and AS/NZS 4600:2018 [2]

Specimen	Nominal thickness (mm)	Angle α (°)	FE results P^{FE} (kN)	Nominal block shear strength R_n (kN)			$\frac{P^{FE}}{R_n^{AISI}}$	$\frac{P^{FE}}{R_n^{Prop.}}$
				AISI	AS/NZS	Proposed		
				S100:2016 (R2020) w/S3-22	4600:2018	Formula		
A2.4-12-00	2.4	0	75.06	73.01	73.35	73.01	1.03	1.03
A2.4-12-04	2.4	4	74.16	74.04	-	74.98	1.00	0.99
A2.4-12-10	2.4	10	75.09	67.39	-	72.50	1.11	1.04
A2.4-12-27	2.4	27	79.12	72.81	-	79.40	1.09	1.00
A2.4-12-37	2.4	37	91.35	85.73	-	90.42	1.07	1.01
A2.4-12-45	2.4	45	107.06	101.24	-	104.14	1.06	1.03
A2.4-08-00	2.4	0	47.16	46.01	46.34	46.01	1.02	1.02
A2.4-08-06	2.4	6	46.71	39.57	-	46.32	1.18	1.01
A2.4-08-13	2.4	13	45.91	38.14	-	45.29	1.20	1.01
A2.4-08-29	2.4	29	50.18	45.01	-	50.65	1.11	0.99
A2.4-08-38	2.4	38	57.09	53.01	-	57.14	1.08	1.00
A2.4-08-45	2.4	45	65.76	62.08	-	64.90	1.06	1.01
A1.9-12-00	1.9	0	59.46	57.73	57.99	57.73	1.03	1.03
A1.9-12-04	1.9	4	58.74	58.65	-	59.28	1.00	0.99
A1.9-12-10	1.9	10	59.01	53.28	-	57.34	1.11	1.03
A1.9-12-27	1.9	27	63.37	57.67	-	62.83	1.10	1.01
A1.9-12-37	1.9	37	72.11	67.93	-	71.58	1.06	1.01
A1.9-12-45	1.9	45	84.58	80.23	-	82.48	1.05	1.03
A1.9-08-00	1.9	0	37.52	36.37	36.63	36.37	1.03	1.03
A1.9-08-06	1.9	6	36.22	31.32		36.62	1.16	0.99
A1.9-08-13	1.9	13	35.85	30.19	-	35.81	1.19	1.00
A1.9-08-29	1.9	29	39.89	35.64	-	40.07	1.12	1.00
A1.9-08-38	1.9	38	45.57	41.99	-	45.23	1.09	1.01
A1.9-08-45	1.9	45	52.35	49.18	-	51.39	1.06	1.02
B2.4-12-00	2.4	0	75.06	73.01	73.35	73.01	1.03	1.03
B2.4-12-08	2.4	8	72.53	67.84	-	72.40	1.07	1.00
B2.4-12-30	2.4	30	73.68	66.35	-	73.85	1.11	1.00
B2.4-12-60	2.4	60	83.17	79.69	-	81.58	1.04	1.02
B2.4-12-80	2.4	80	87.35	85.02	-	84.51	1.03	1.03
B2.4-12-90	2.4	90	87.24	82.39	82.51	82.39	1.06	1.06
B2.4-08-00	2.4	0	47.16	46.01	46.34	46.01	1.02	1.02
B2.4-08-08	2.4	8	45.72	39.20	-	45.40	1.17	1.01

B2.4-08-30	2.4	30	45.07	39.91	-	45.76	1.13	0.98
B2.4-08-60	2.4	60	50.51	48.87	-	50.11	1.03	1.01
B2.4-08-80	2.4	80	52.85	52.37	-	51.78	1.01	1.02
B2.4-08-90	2.4	90	52.84	50.95	51.07	50.95	1.04	1.04
B1.9-12-00	1.9	0	59.46	57.73	57.99	57.73	1.03	1.03
B1.9-12-08	1.9	8	57.61	53.72	-	57.25	1.07	1.01
B1.9-12-30	1.9	30	58.07	52.55	-	58.45	1.11	0.99
B1.9-12-60	1.9	60	66.09	63.13	-	64.62	1.05	1.02
B1.9-12-80	1.9	80	69.40	67.36	-	66.95	1.03	1.04
B1.9-12-90	1.9	90	69.18	65.27	65.37	65.27	1.06	1.06
B1.9-08-00	1.9	0	37.52	36.37	36.63	36.37	1.03	1.03
B1.9-08-08	1.9	8	35.88	31.03	-	35.90	1.16	1.00
B1.9-08-30	1.9	30	35.50	31.60	-	36.21	1.12	0.98
B1.9-08-60	1.9	60	39.96	38.70	-	39.68	1.03	1.01
B1.9-08-80	1.9	80	41.63	41.48	-	41.01	1.00	1.02
B1.9-08-90	1.9	90	41.54	40.35	40.45	40.35	1.03	1.03
C2.4-12-35	2.4	35	55.90	49.49	-	56.88	1.13	0.98
C2.4-12-50	2.4	50	60.63	56.50	-	61.00	1.07	0.99
C2.4-12-65	2.4	65	64.75	62.49	-	64.38	1.04	1.01
C2.4-12-75	2.4	75	66.42	65.19	-	65.89	1.02	1.01
C2.4-12-90	2.4	90	67.29	64.51	64.51	64.51	1.04	1.04
C2.4-08-35	2.4	35	36.17	30.69	-	36.50	1.18	0.99
C2.4-08-50	2.4	50	38.64	35.41	-	38.86	1.09	0.99
C2.4-08-65	2.4	65	40.83	39.34	-	40.78	1.04	1.00
C2.4-08-75	2.4	75	41.84	41.12	-	41.65	1.02	1.00
C2.4-08-90	2.4	90	42.41	40.89	40.89	40.89	1.04	1.04
C1.9-12-35	1.9	35	44.11	39.16	-	44.99	1.13	0.98
C1.9-12-50	1.9	50	47.78	44.72	-	48.27	1.07	0.99
C1.9-12-65	1.9	65	51.19	49.48	-	50.97	1.03	1.00
C1.9-12-75	1.9	75	52.49	51.62	-	52.17	1.02	1.01
C1.9-12-90	1.9	90	53.28	51.08	51.08	51.08	1.04	1.04
C1.9-08-35	1.9	35	28.65	24.28	-	28.86	1.18	0.99
C1.9-08-50	1.9	50	30.47	28.03	-	30.74	1.09	0.99
C1.9-08-65	1.9	65	32.17	31.14	-	32.28	1.03	1.00
C1.9-08-75	1.9	75	32.93	32.55	-	32.97	1.01	1.00
C1.9-08-90	1.9	90	33.55	32.37	32.37	32.37	1.04	1.04
D2.4-12-39	2.4	39	75.46	63.26	-	77.74	1.19	0.97
D2.4-12-50	2.4	50	80.90	73.03	-	82.72	1.11	0.98
D2.4-12-65	2.4	65	88.13	84.20	-	88.27	1.05	1.00
D2.4-12-75	2.4	75	91.41	89.25	-	90.78	1.02	1.01
D2.4-12-90	2.4	90	92.17	88.76	88.76	88.76	1.04	1.04

D2.4-08-39	2.4	39	46.90	36.43	-	47.43	1.29	0.99
D2.4-08-50	2.4	50	50.03	42.98	-	50.20	1.16	1.00
D2.4-08-65	2.4	65	52.94	50.30	-	53.29	1.05	0.99
D2.4-08-75	2.4	75	54.82	53.52	-	54.62	1.02	1.00
D2.4-08-90	2.4	90	55.09	53.57	53.57	53.57	1.03	1.03
D1.9-12-39	1.9	39	59.75	50.09	-	61.52	1.19	0.97
D1.9-12-50	1.9	50	64.08	57.85	-	65.49	1.11	0.98
D1.9-12-65	1.9	65	69.30	66.71	-	69.92	1.04	0.99
D1.9-12-75	1.9	75	71.96	70.71	-	71.92	1.02	1.00
D1.9-12-90	1.9	90	72.85	70.33	70.33	70.33	1.04	1.04
D1.9-08-39	1.9	39	36.98	28.83	-	37.52	1.28	0.99
D1.9-08-50	1.9	50	39.34	34.02	-	39.74	1.16	0.99
D1.9-08-65	1.9	65	41.82	39.84	-	42.20	1.05	0.99
D1.9-08-75	1.9	75	43.27	42.40	-	43.26	1.02	1.00
D1.9-08-90	1.9	90	43.56	42.43	42.43	42.43	1.03	1.03
Mean							1.08	1.01
C.o.V (%)							5.78	1.97

6.3.2. Bolted connections under curling effect

6.3.2.1 Input parameters

In this section, a Finite Element Model (FEM) numerical simulation is developed and subsequently validated against the test results using the commercial program ABAQUS/Explicit simulation. A typical FEM is demonstrated in Figure 6.22 with actual material properties as the input into the model extracted from the coupon tests (Table 5.1) after taking the transformation (according to Equations (6.2) and (6.3)) of the nominal stress and strain to the true stress and true plastic strain (see Figure 6.23). In addition, material properties of the G300 thick packing plates between CFS sheets are extracted from the coupon tests in Appendix B.

In the simulation, a through thickness isotropic material behaviour (i.e., the plane through thickness direction is the plane of isotropy at every point in the material) in conjunction with the ductile damage model are applied based on the concept in the previous sections. The friction coefficient between the cold-formed sheets and the hot-rolled sheets is assumed to be 0.3 whereas that between the bolt threads and the holes is chosen as 1.2 in order to hold the bolts in place. A load rate of 210 mm/min is employed to obtain a suitable response with insignificant effect of kinetic energy. It should be noted that the loading rate of the FEM does not affect results since the quasi-static material properties are input to the simulation.

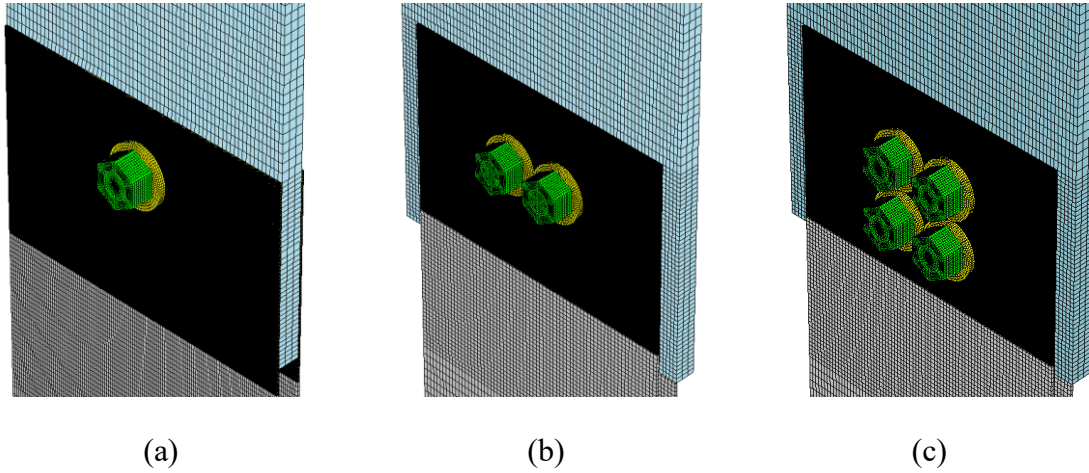


Figure 6.22: Schematic of typical Finite Element Modelling of the bolted connection (a) Configuration Type 1, (b) Configuration Type 2 and (c) Configuration Type 3. Note: a load speed of 210 mm/min was applied to the top hot-rolled plate while fixed-end boundary condition was applied to the bottom hot-rolled plate, which was similar to the corresponding experimental program

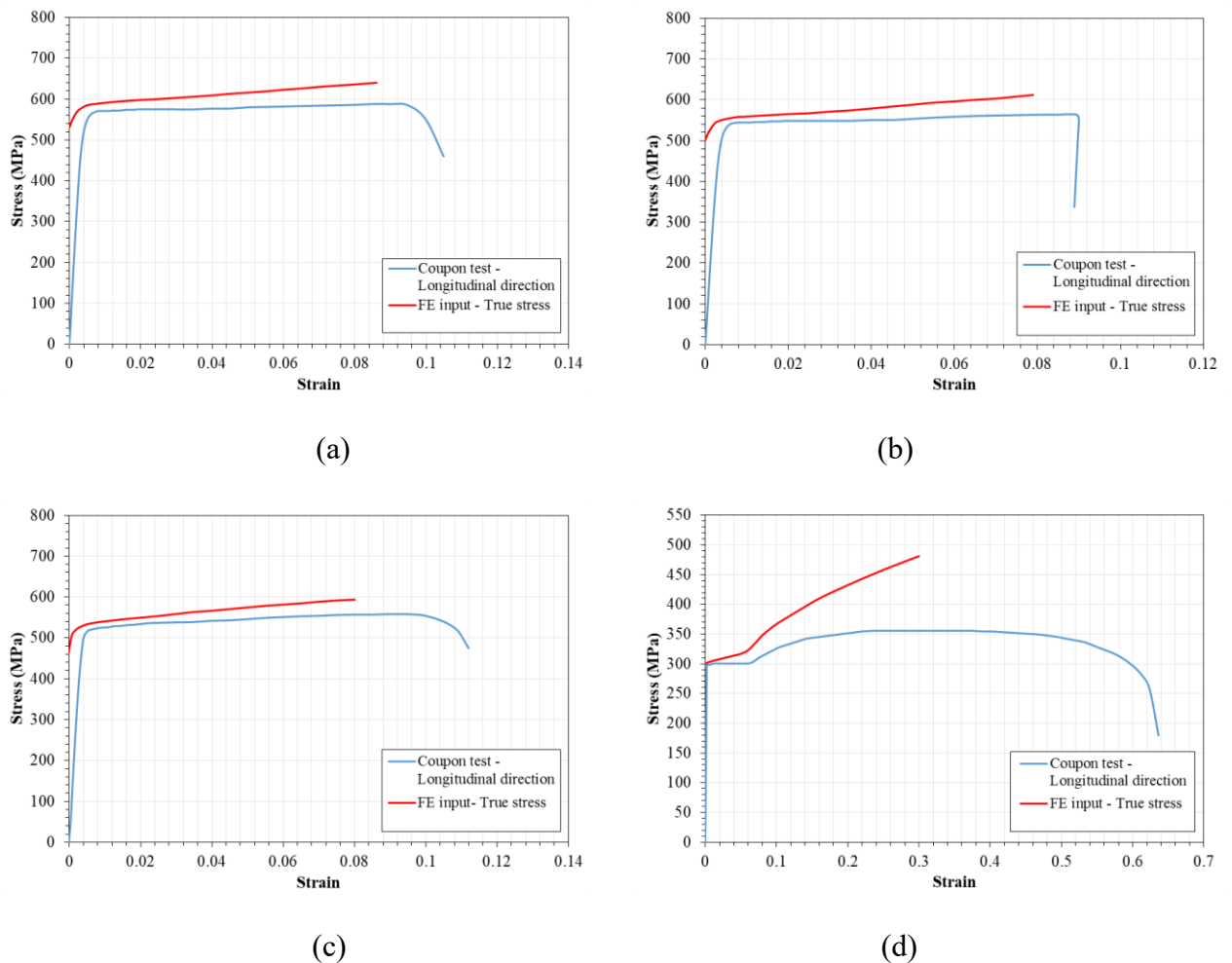


Figure 6.23: Stress-strain curves from coupon tests in longitudinal direction and FE input for (a) 1.0mm G550 cold-formed sheets, (b) 1.2mm G500 cold-formed sheets, (c) 1.9mm G450 cold-formed sheets and (d) 1.9mm G300 cold-formed sheets

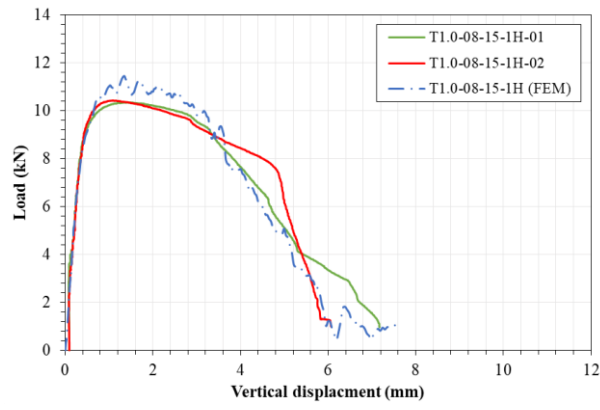
6.3.2.2 Finite element results

Table 6.10 illustrates the FE results in comparison with the experimental results of the bolted connections. It is obvious that all the FE results are approximately similar with the test results through the average ratios of FE results divided by experimental results and their corresponding coefficient of variation (i.e., Mean = 1.05 and C.o.V = 3.21%) although almost FE results are slightly higher than the experimental results due to the imperfections in the actual tests (i.e., homogeneous material of sheet steels, symmetric conditions of test set up, etc.), which verifies the validation of the calibrated FEM. The validation of the current simulation is further confirmed based on the relationship between the displacement and the shear strength shown in Figure 6.24 - Figure 6.27. Overall, the load-displacement FEM curves are quite consistent and achieve good agreement with the experiments' ones.

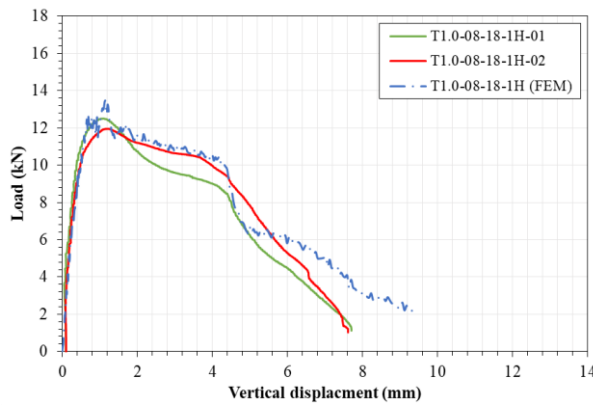
The failure process of the specimen T1.2-12-30-2H under the presence of the curling is compared between the FEM simulation and the experiment, as demonstrated in Figure 6.28. Clearly, the FEM simulation provides good agreement with the test results through 6 stages of the fracture process. The shear strength of the connection increases dramatically in the elastic phase (e.g., Stage 1) before curling occurs at the free edge (e.g., Stage 2). After that, the shear strength achieves a peak (e.g., Stage 3) in a short time before a sudden strength reduction appears in Stage 4 with a large out-of-plane displacement, followed by tensile fracture and shear fracture in Stage 5 and Stage 6, respectively.

Table 6.10: Comparison of the FE results with the experimental results of the bolted connections

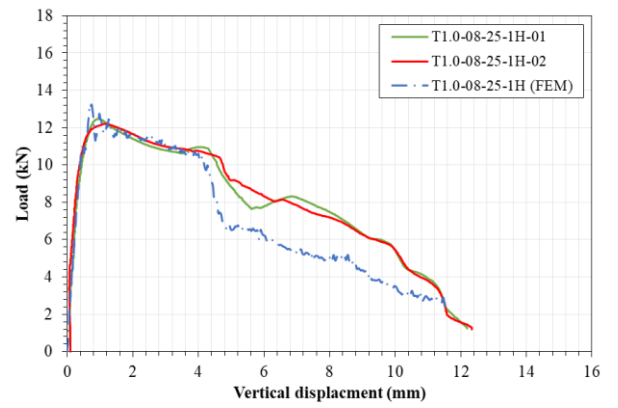
Config.	Specimen	Nominal thickness t (mm)	Nominal hole size d_h (mm)	End distance e_1 (mm)	Average test results P^T (kN)	FE results P^{FE} (kN)	Ratio $\frac{P^{FE}}{P^T}$
Type 1	T1.0-08-15-1H	1.00	9	15	10.40	11.45	1.10
Type 1	T1.0-08-18-1H	1.00	9	18	12.23	13.48	1.10
Type 1	T1.0-08-25-1H	1.00	9	25	12.35	13.24	1.07
Type 1	T1.0-12-19-1H	1.00	14	19	12.77	13.36	1.05
Type 1	T1.0-12-22-1H	1.00	14	22	14.62	14.98	1.02
Type 1	T1.0-12-30-1H	1.00	14	30	14.03	13.90	0.99
Type 2	T1.2-08-17-2H	1.20	9	17	20.47	22.14	1.08
Type 2	T1.2-08-20-2H	1.20	9	20	22.55	24.96	1.11
Type 2	T1.2-08-25-2H	1.20	9	25	27.10	28.62	1.06
Type 2	T1.2-12-18-2H	1.20	14	18	22.92	23.77	1.04
Type 2	T1.2-12-22-2H	1.20	14	22	26.02	27.85	1.07
Type 2	T1.2-12-25-2H	1.20	14	25	28.87	30.86	1.07
Type 2	T1.2-12-30-2H	1.20	14	30	30.33	30.62	1.01
Type 3	T1.2-08-17-4H	1.20	9	17	34.64	35.12	1.01
Type 3	T1.2-08-20-4H	1.20	9	20	38.44	37.74	0.98
Type 3	T1.2-12-22-4H	1.20	14	22	41.43	42.61	1.03
Type 3	T1.2-12-25-4H	1.20	14	25	43.28	45.11	1.04
Type 3	T1.9-08-28-4H	1.90	9	28	59.39	62.47	1.05
Type 3	T1.9-08-32-4H	1.90	9	32	64.74	67.28	1.04
Type 3	T1.9-08-35-4H	1.90	9	35	64.43	68.49	1.06
Type 3	T1.9-08-40-4H	1.90	9	40	63.36	67.88	1.07
Type 3	T1.9-12-25-4H	1.90	14	25	41.80	44.23	1.06
Type 3	T1.9-12-35-4H	1.90	14	35	48.70	52.21	1.07
Type 3	T1.9-12-42-4H	1.90	14	42	50.52	55.29	1.09
						Mean	1.05
						C.o.V (%)	3.21



(a)

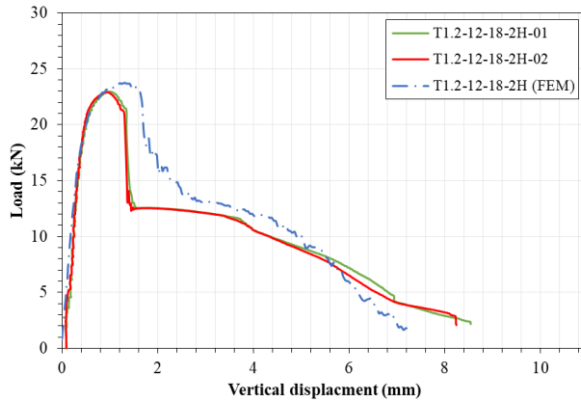


(b)

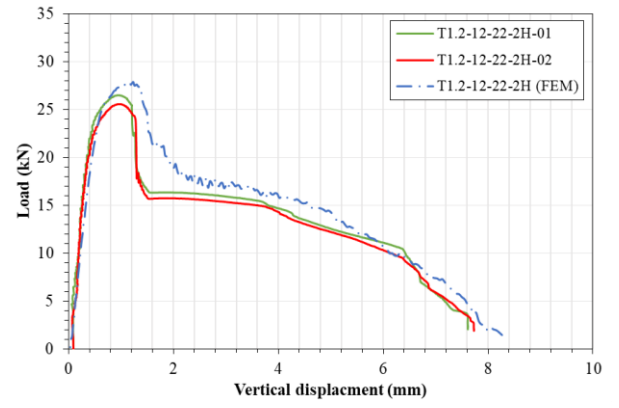


(c)

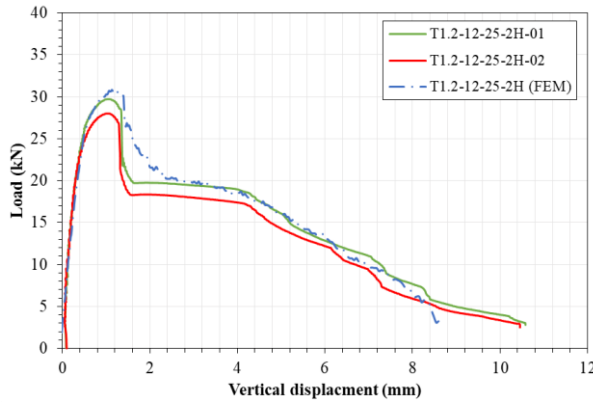
Figure 6.24: Comparison of the FE results with the experiments of the 1.0mm G550 cold-formed steel bolted connections Configuration Type 1 using M8-bolts (a) with the end distance $e_1 = 15$, (b) with the end distance $e_1 = 18$ and (c) with the end distance $e_1 = 25$



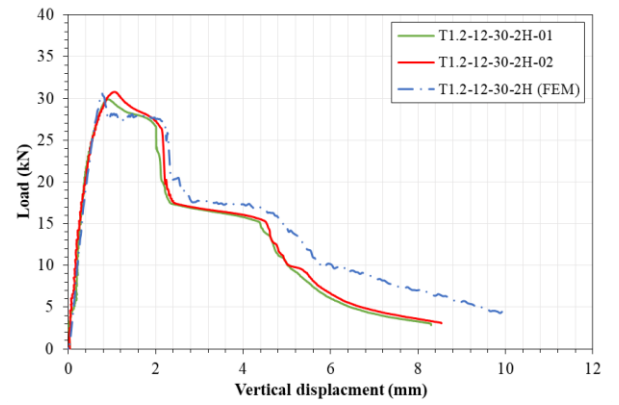
(a)



(b)

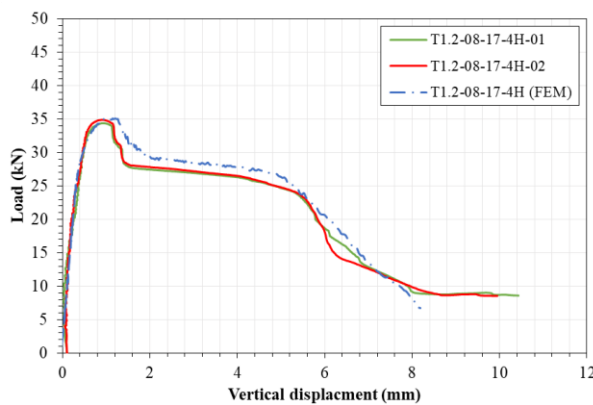


(c)

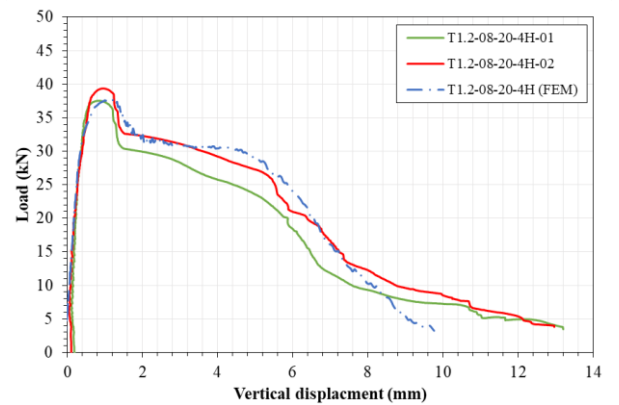


(d)

Figure 6.25: Comparison of the FE results with the experiments of the 1.2mm G500 cold-formed steel bolted connections Configuration Type 2 using M12-bolts (a) with the end distance $e_1 = 18$, (b) with the end distance $e_1 = 22$, (c) with the end distance $e_1 = 25$ and (d) with the end distance $e_1 = 30$

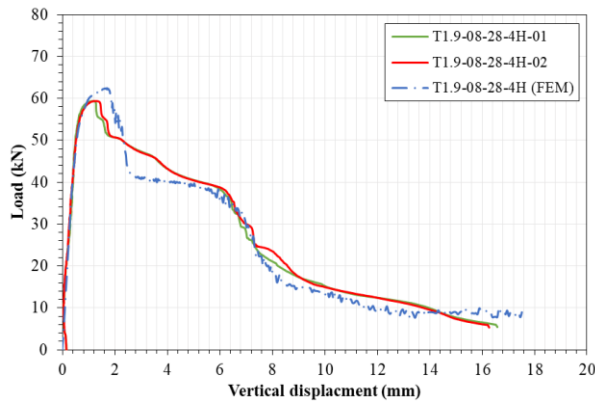


(a)

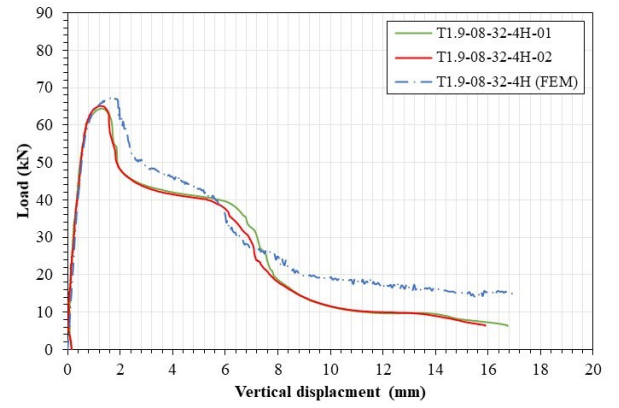


(b)

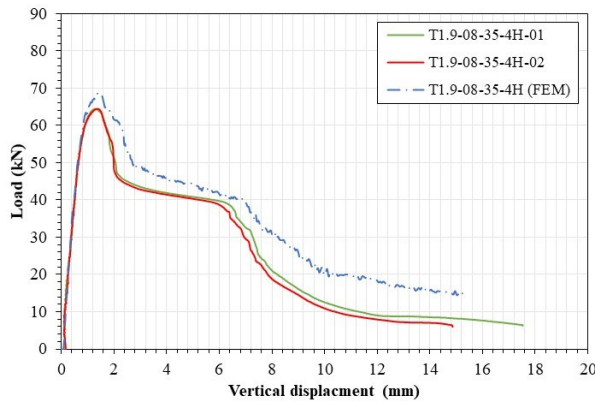
Figure 6.26: Comparison of the FE results with the experiments of the 1.2mm G500 cold-formed steel bolted connections Configuration Type 3 using M8-bolts (a) with the end distance $e_1 = 17$ and (b) with the end distance $e_1 = 20$



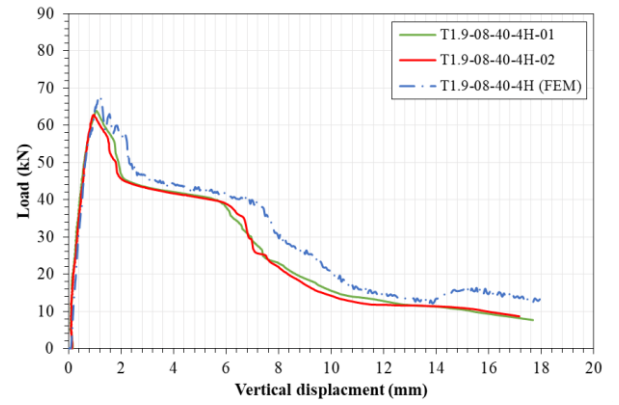
(a)



(b)



(c)



(d)

Figure 6.27: Comparison of the FE results with the experiments of the 1.9mm G450 cold-formed steel bolted connections Configuration Type 3 using M8-bolts (a) with the end distance $e_1 = 28$, (b) with the end distance $e_1 = 32$, (c) with the end distance $e_1 = 35$ and (d) with the end distance $e_1 = 40$

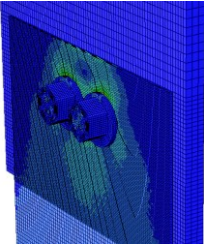
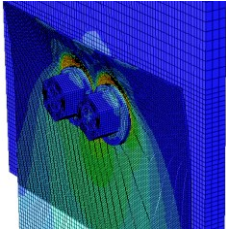
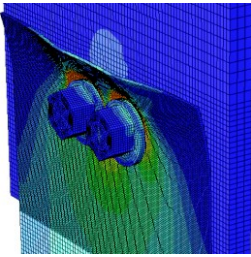
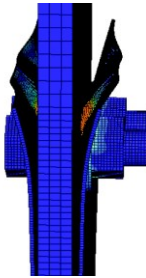
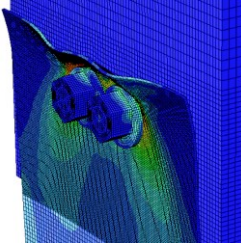
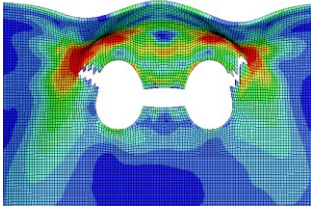
Experiment	FE modelling
	
Stage 1	Stage 1
	
Stage 2	Stage 2
	
Stage 3	Stage 3
	
Stage 4	Stage 4
	
Stage 5	Stage 5
	
Stage 6	Stage 6

Figure 6.28: Comparison of failure process of the specimen T1.2-12-30-2H between the FEM and the experiment

6.3.2.3 Parametric study

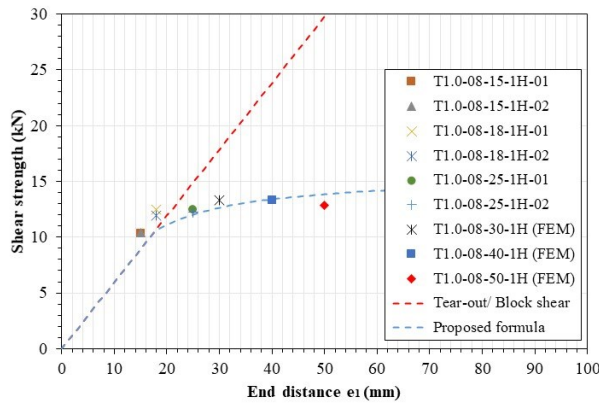
The validated FEM simulation consequently plays a crucial role to extend the data range for comparison with the proposed design Equation (5.8), as shown in Table 6.11. In particular, the ratio of FE results divided by the predicted values ranges from 0.92 to 1.30. It is worth pointing out that the proposed design Equation (5.8) provides a good agreement with the calibrated FE results via the average ratio $P^{FE}/R_n^{Prop.}$ and the corresponding coefficient of variation (i.e., Mean = 1.11 and C.o.V = 9.54%).

In addition, the accuracy of the proposed design Equation (5.8) is further compared with the experimental and calibrated FE results in conjunction with the current design formulas from AS/NZS 4600:2018 [2] as shown in Figure 6.29 - Figure 6.32. It is obvious that the proposed formula is more accurate to predict the nominal shear strength with the presence of the curling. In particular, the nominal shear strength determined by the design formulas from AS/NZS 4600:2018 [2] increases monotonically without consideration of the curling while that from the proposal keeps nearly constant and converges to an ultimate value when the curling is starting to occur, as observed from both the experiments and FEM. Therefore, the AS/NZS 4600:2018 [2] predictions overestimate the nominal shear strength against the experimental and calibrated FE results in comparison with the proposed formula (5.8).

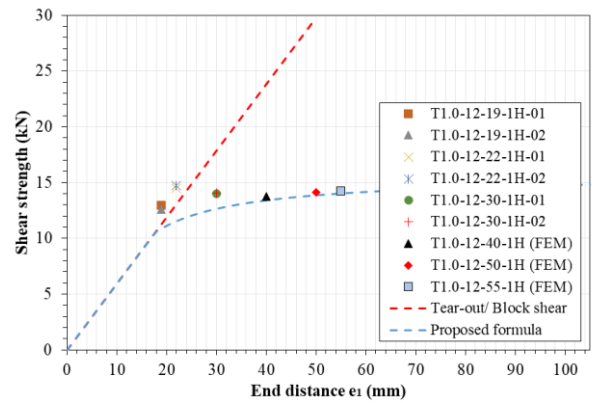
Table 6.11: Comparison of the calibrated FE results with the proposed design Equation (5.8) for the cold-formed steel bolted connections

Config.	Specimen	Nominal thickness t (mm)	Nominal hole size d_h (mm)	End distance e_1 (mm)	Resistant length e_{rs} (mm)	FE results P^{FE} (kN)	Proposed formula $R_n^{Prop.}$ (kN)	Ratio $\frac{P^{FE}}{R_n^{Prop.}}$
Type 1	T1.0-08-15-1H	1.00	9.0	15	15.0	11.45	8.93	1.28
Type 1	T1.0-08-18-1H	1.00	9.0	18	17.8	13.48	10.63	1.27
Type 1	T1.0-08-25-1H	1.00	9.0	25	20.2	13.24	12.04	1.10
Type 1	T1.0-08-30-1H	1.00	9.0	30	21.2	13.30	12.64	1.05
Type 1	T1.0-08-40-1H	1.00	9.0	40	22.5	13.31	13.39	0.99
Type 1	T1.0-08-50-1H	1.00	9.0	50	23.3	12.88	13.85	0.93
Type 1	T1.0-12-19-1H	1.00	14.0	19	18.3	13.36	10.89	1.23
Type 1	T1.0-12-22-1H	1.00	14.0	22	19.4	14.98	11.54	1.30
Type 1	T1.0-12-30-1H	1.00	14.0	30	21.2	13.90	12.64	1.10
Type 1	T1.0-12-40-1H	1.00	14.0	40	22.5	13.77	13.39	1.03
Type 1	T1.0-12-50-1H	1.00	14.0	50	23.3	14.12	13.85	1.02
Type 1	T1.0-12-55-1H	1.00	14.0	55	23.5	14.26	14.01	1.02
Type 2	T1.2-08-17-2H	1.20	9.0	17	17.0	22.14	17.60	1.26
Type 2	T1.2-08-20-2H	1.20	9.0	20	20.0	24.96	19.99	1.25
Type 2	T1.2-08-25-2H	1.20	9.0	25	22.9	28.62	22.31	1.28
Type 2	T1.2-08-35-2H	1.20	9.0	35	25.5	29.39	24.34	1.21
Type 2	T1.2-08-40-2H	1.20	9.0	40	26.3	29.21	24.97	1.17
Type 2	T1.2-08-50-2H	1.20	9.0	50	27.4	29.94	25.86	1.16

Type 2	T1.2-08-60-2H	1.20	9.0	60	28.1	30.94	26.45	1.17
Type 2	T1.2-12-18-2H	1.20	14.0	18	18.0	23.77	20.67	1.15
Type 2	T1.2-12-22-2H	1.20	14.0	22	21.7	27.85	23.61	1.18
Type 2	T1.2-12-25-2H	1.20	14.0	25	22.9	30.86	24.58	1.26
Type 2	T1.2-12-30-2H	1.20	14.0	30	24.4	30.62	25.76	1.19
Type 2	T1.2-12-40-2H	1.20	14.0	40	26.3	32.23	27.24	1.18
Type 2	T1.2-12-50-2H	1.20	14.0	50	27.4	32.44	28.12	1.15
Type 2	T1.2-12-60-2H	1.20	14.0	60	28.1	31.30	28.72	1.09
Type 3	T1.2-08-17-4H	1.20	9.0	17	17.0	35.12	28.32	1.24
Type 3	T1.2-08-20-4H	1.20	9.0	20	20.0	37.74	30.70	1.23
Type 3	T1.2-08-30-4H	1.20	9.0	30	24.4	37.29	34.21	1.09
Type 3	T1.2-08-40-4H	1.20	9.0	40	26.3	36.64	35.69	1.03
Type 3	T1.2-08-50-4H	1.20	9.0	50	27.4	37.13	36.57	1.02
Type 3	T1.2-08-55-4H	1.20	9.0	55	27.8	35.84	36.90	0.97
Type 3	T1.2-12-22-4H	1.20	14.0	22	21.7	42.61	40.28	1.06
Type 3	T1.2-12-25-4H	1.20	14.0	25	22.9	45.11	41.25	1.09
Type 3	T1.2-12-35-4H	1.20	14.0	35	25.5	46.16	43.28	1.07
Type 3	T1.2-12-45-4H	1.20	14.0	45	26.9	45.32	44.40	1.02
Type 3	T1.2-12-55-4H	1.20	14.0	55	27.8	43.85	45.12	0.97
Type 3	T1.9-08-28-4H	1.90	9.0	28	28.0	62.47	57.77	1.08
Type 3	T1.9-08-32-4H	1.90	9.0	32	32.0	67.28	62.72	1.07
Type 3	T1.9-08-35-4H	1.90	9.0	35	34.6	68.49	65.93	1.04
Type 3	T1.9-08-40-4H	1.90	9.0	40	36.6	67.88	68.45	0.99
Type 3	T1.9-08-50-4H	1.90	9.0	50	39.5	68.04	71.97	0.95
Type 3	T1.9-08-60-4H	1.90	9.0	60	41.4	69.57	74.31	0.94
Type 3	T1.9-08-70-4H	1.90	9.0	70	42.7	69.95	75.99	0.92
Type 3	T1.9-12-25-4H	1.90	14.0	25	25.0	44.23	37.85	1.17
Type 3	T1.9-12-35-4H	1.90	14.0	35	35.0	52.21	44.62	1.17
Type 3	T1.9-12-42-4H	1.90	14.0	42	42.0	55.29	49.36	1.12
Type 3	T1.9-12-60-4H	1.90	14.0	60	51.4	57.12	55.73	1.02
Type 3	T1.9-12-70-4H	1.90	14.0	70	53.9	59.81	57.41	1.04
Type 3	T1.9-12-85-4H	1.90	14.0	85	56.5	60.99	59.18	1.03
Mean								1.11
C.o.V (%)								9.54

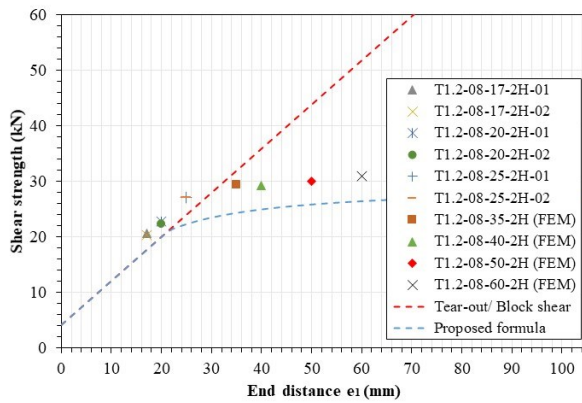


(a)

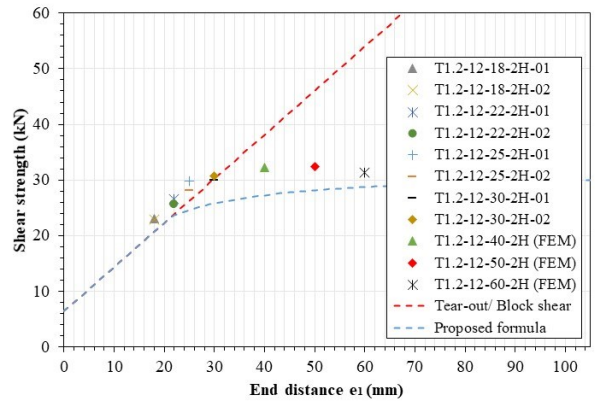


(b)

Figure 6.29: Comparison of the experimental and calibrated FE results with the proposed design Equation (5.8) in conjunction with the current design formulas from AS/NZS 4600:2018 [2] of the 1.0mm G550 cold-formed steel bolted connections Configuration Type 1 using (a) M8-bolts and (b) M12-bolts

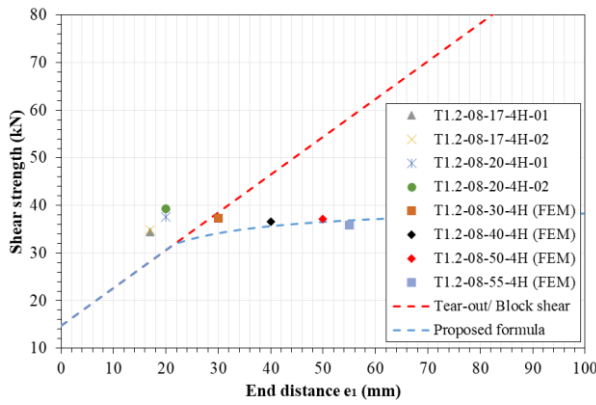


(a)

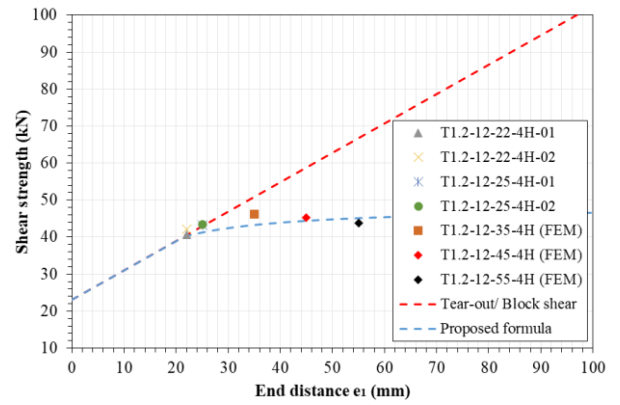


(b)

Figure 6.30: Comparison of the experimental and calibrated FE results with the proposed design Equation (5.8) in conjunction with the current design formulas from AS/NZS 4600:2018 [2] of the 1.2mm G500 cold-formed steel bolted connections Configuration Type 2 using (a) M8-bolts and (b) M12-bolts

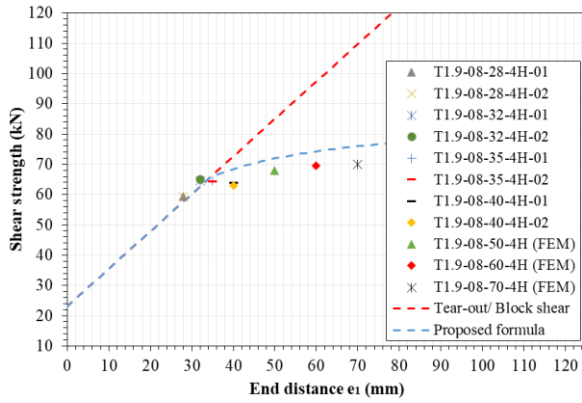


(a)

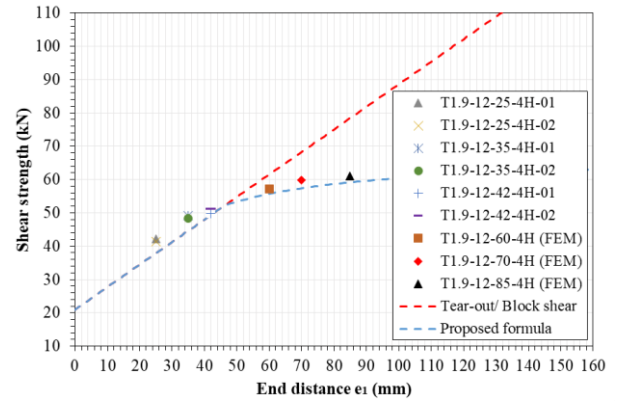


(b)

Figure 6.31: Comparison of the experimental and calibrated FE results with the proposed design Equation (5.8) in conjunction with the current design formulas from AS/NZS 4600:2018 [2] of the 1.2mm G500 cold-formed steel bolted connections Configuration Type 3 using (a) M8-bolts and (b) M12-bolts



(a)



(b)

Figure 6.32: Comparison of the experimental and calibrated FE results with the proposed design Equation (5.8) in conjunction with the current design formulas from AS/NZS 4600:2018 [2] of the 1.9mm (a) G450 cold-formed steel bolted connections Configuration Type 3 using M8-bolts and (b) G300 cold-formed steel bolted connections Configuration Type 3 using M12-bolts

6.4. SUMMARY

The validated FE models have been successfully generated and verified against the experimental results using the ABAQUS software. Numerical analyses are conducted using different material models, fracture criteria to capture the most accurate and economic results. The FE models provide detailed insights which could not be seen in actual tests (i.e., stress and strain distributions). In addition, the validated FE models are used successfully to investigate a wide range of the corresponding experiments as a parametric study to achieve reliable data. These

datasets further confirm the effectiveness and accuracy of the proposed design formulas in the previous sections.

CHAPTER 7

CONCLUSIONS

7.1. CONCLUDING REMARKS

This thesis addresses the fracture behaviour and mainly proposes new design guidelines for bolted connections in high strength CFS plates and channels. Chapters 1 and 2 describe the background, literature, research objectives, research methodologies and the contributions of this thesis and its relevant hypotheses. Chapters 3 through to 6 present experimental, numerical, and analytical research, which was conducted to achieve the research objectives. The key findings of the particular chapters are summarised below.

The fracture behaviour and block shear capacity of bolted connections with staggered holes subjected to monotonic loading were investigated in Chapter 3 based on an experimental program comprised of 24 tests. The cold-formed steel sheets included two thicknesses (i.e., 1.9mm and 2.4mm) using G450 sheet steel to AS1397. High strength steel bolts (i.e., Grade 8.8) with two diameters (i.e., 8mm and 12mm) were used in the tests in order to eliminate the shear failure in the bolts, and to investigate the influence of hole size on the staggered hole bolted connections. The test apparatus used conformed with the AISI S905-13 Standard and the AISI Specification-RP01-4 so that the test results could be used to examine the current design equations in the AISI S100:16 (R2020) Specification with the S3-22 Supplement and the AS/NZS 4600:2018 Standard. The test results provide valuable insights into the fracture behaviour of the bolted connections under the presence of the staggered holes. The inclined paths between the staggered holes not only affected the block shear strength but also changed the fracture behaviour of the bolted connections. Furthermore, the presence of the staggered holes also reduced the ductility of the bolted connections in comparison with traditional rectangular block shear connections. The fracture paths in the inclined zones were shown to correlate clearly with the angle α instead of occurring in the centerline between the staggered holes. The nominal shear strength in the staggered paths correlated with the angle α including two components which are the shear strength on the failure path parallel to the staggered path and the tension strength on the failure path perpendicular to the staggered path. Finally, a design equation for block shear strength with staggered holes is proposed, which provides a better correlation with the experimental results than current design standards and specifications. It is currently under ballot by the AISI Specification committee. In addition, the current AISI S100 predictions are more conservative in obtaining the nominal block shear strength against the test results in comparison with the proposed formula while AS/NZS 4600 does not include specific equations for staggered block shear failures. With a

target reliability index (β) of 3.5, the reliability analysis demonstrates that the reliability indices (i.e., resistance factor $\phi = 0.82$ and safety factor $\Omega = 1.77$) are better than the current values (resistance factor $\phi = 0.75$ and safety factor $\Omega = 2.00$) used for block shear rupture of CFS staggered hole bolted connections.

Chapter 4 investigated the complex fracture behaviour of shear beam-end bolted connections failing in the block shear mode under the combined effects of shear and rotation through a series of 78 tests. In particular, the AISI S905-13, AS 4100:2020 Standards were employed as the testing methodologies. In addition, the CFS channels were made from high-strength cold-reduced sheet steel to AS 1397:2021 whereas the T-shaped sections to which the sections were bolted were made from hot-rolled steel plates to AS/NZS 3678:2016. All the bolts were high-strength bolts (Grade 8.8) to AS/NZS 1252.1:2016. The CFS channel thicknesses of 1.2mm and 1.5mm in conjunction with various shear distances between the bolts were the main variables used to evaluate the effect of the shear fracture paths and fracture process behaviour on the shear rupture strength of the connections. As determined from the experiments, the rotation of the CFS channels only provided a minor effect on the block shear capacity and fracture behaviour of the shear beam-end bolted connections because of the high difference between the bolt size and hole size causing a high initial slip and rotation of the connections during loading. However, the rotation had significantly affected the shear strength of the screw connections (Huynh et al. 2021) and shear strength of the PAF connections (Truong et al. 2022) where bearing failure dominated the failure mode in the same test rig. In addition, the shear capacity of the bolted connections failing in block shear only depended on the fracture paths which did not change in the case of bolted connections using running ratio (RR=2.0) (e.g., the ratio of speed of Actuator A over the speed of Actuator B) whereas the connections not only provided a different block shear limit state (i.e., shear fracture occurs along different paths) but also showed a large deformation when using a higher running ratio (RR=3.0). The latter was not appropriate for practical design. However, the block shear capacity and fracture behaviour in the shear zones of the shear beam-end bolted connections also significantly depended on the shear distance (s) between the bolts. In particular, the shear fracture and tensile fracture of the connections appeared simultaneously after reaching the peak when using a relatively short shear distance (s) because the ultimate tensile stress in the tension zone and the ultimate shear stress in the shear zone between the two bolts localized simultaneously. On the other hand, a reverse behaviour was apparent in the connections using a relatively long shear distance (s). The ultimate tensile stress localized around the tension zone while the shear stress in the shear zone between the two bolts did not achieve its ultimate value. In this situation, after the tensile fracture occurred at the bottom bolt in Region 3, shear yielding started to occur in the shear zone between the two bolts in Region 2. It was also observed that the

active shear planes also occurred during the testing of the bolted shear connections. Comparison of the test results with the design standards and specifications from AISI S100:16 (R2020) w/S3-22 and AS/NZS 4600:2018 demonstrates that although the current design equations in the standards/specifications predict accurately the nominal block shear capacity of the shear beam-end bolted connections in general, that predictions are more conservative in obtaining the nominal block shear strength against the experimental results in specific cases where the edge distance (e_2) is relatively long while the shear distance (s) is relatively short. Therefore, the AISI S100:16 (R2020) w/S3-22 and AS/NZS 4600:2018 predictions are inaccurate to obtain the nominal block shear capacity without considering the effect of the shear distance (s) on the fracture process behaviour of the connections as observed from the experiments. According to the reliability analysis, a resistance factor ($\phi = 0.90$) can be used for rupture of the bolted connections with a target reliability index (β) of 3.5 for $D/L = 0.2$. This value is significantly higher than that employed by AS/NZS 4600:2018 ($\phi = 0.80$) and AISI S100:2016 (R2020) w/S3-22 ($\phi = 0.75$) because of the good quality control of the materials involved in the tests and the reliability of the tests.

The fracture behaviour and block shear and tear-out capacities of the bolted connections between thin and thick plates undergoing the effect of the curling phenomenon of the thin plate were investigated in Chapter 5 based on a series of 48 tests. The test apparatus used conformed with the AISI S905-13 Standard and the AISI Specification-RP01-4. In the experimental program, different material properties of the cold-formed sheet steels such as G300, G450, G500 and G550 to AS 1397-2021 were used for the specimens to create a large database for the investigation. The cold-formed steel sheets with thicknesses of 1.0mm, 1.2mm and 1.9mm in conjunction with two different hole diameters (9mm and 14mm) corresponding to high strength steel Grade 8.8 bolts (i.e., 8mm and 12mm) and various end distances as the main variables were studied to investigate the curling effect. Three different failure modes being block shear, tear-out and curling were observed in the tests. The test results demonstrated that curling not only affected the shear strength but also changed the fracture behaviour of the bolted connections. A typical tear-out or block shear fracture occurred without curling phenomenon in the bolted connections when using a relatively short end distance whereas the curling phenomenon appeared in the bolted connections when using a relatively long end distance causing a sudden strength reduction in the load-displacement curves. As a result, the shear strength achieved an ultimate value and kept constant although the end distance continued to increase. Furthermore, the hole size, thickness and grade of steel of the cold-formed plates of the connections affected the curling behaviour. For example, larger hole sizes, thicker plates and higher grades of steel are less likely to curl than smaller hole sizes, thinner plates and lower grades of steel, respectively. As investigated from the experimental

and calibrated FE results, the curling phenomenon also changed the stress distribution of the cold-formed plates where only a small area around the holes remained in-plane while the top curled area lost its shear strength in the connection. Finally, the curling was demonstrated to be a direct consequence of the induced transverse compression in the outstanding piece of plate causing bending. Based on the effect of the curling phenomenon investigated in the study, a new design equation for the block shear and tear-out strength accounting for the curling effect is proposed. The proposal implies that only the resistant areas remain to contribute to the shear strength of the connections whereas the curled areas do not add to the shear strength of the connections resulting in no improvement on the shear strength with longer end distances. By comparison with the dataset from the experiments and FE models, the current design equations from AISI S100:16 (R2020) w/S3-22 and AS/NZS 4600:2018 overestimate the nominal shear strength without the consideration of the curling effect. On the other hand, based on the buckling theory and effective width concept, the proposed formula reflects very well the actual behaviour instead of adding an influence coefficient to represent the curling effect as proposed by Kim et al. [3] and Kim and Kuwamura [4]. Consequently, an exact value of the end distance when the curling starts to occur (i.e., e_{rs}) is determined so that the shear strength of the bolted connections is eventually evaluated using the resistant length (e_{rs}) instead of the end distance (e_1) as used in AISI S100:16 (R2020) w/S3-22 and AS/NZS 4600:2018. With a target reliability index (β) of 3.5, the reliability analysis demonstrates that the reliability indices (i.e., resistance factor $\phi = 0.86$ and safety factor $\Omega = 1.68$) under the effect of curling are better than the current values (resistance factor $\phi = 0.75$ and safety factor $\Omega = 2.00$) used for block shear and tear-out ruptures of the CFS bolted connections.

Numerical studies in Chapter 6 further investigated the fracture behaviour and shear capacity of the bolted connections against the corresponding previous experiments. The FE models incorporated calibrated fracture parameters of the steel plates to create realistic deformations and damage conditions in the inelastic regime. In particular, a through thickness isotropic material behaviour (i.e., the plane through thickness direction is the plane of isotropy at every point in the material) was applied for the elastic material model that reflected approximately the actual behaviour of the steel sheets after experiencing a longitudinal rolling process. By using the ABAQUS/Explicit solver, the FE models were found to be robust to handle the convergence difficulties caused by large deformations and contact complexity. The FE models were calibrated against the current test results which demonstrated good agreement in terms of initial stiffness, ultimate shear strength and fracture behaviour. The calibrated FE results provided detailed insights which could not be seen in actual tests (i.e., stress and strain distributions) and further

confirmed the effectiveness and accuracy of the proposed design formulas based on the reliable datasets.

7.2. LIMITATIONS AND SUGGESTIONS FOR FUTURE RESEARCH

This research provides valuable insights into the fracture behaviours and shear capacity of bolted connections which can be added to the literature and design provisions. However, the scope of this study has certain limitations which are discussed below.

Throughout the research, the bolts and steel materials are limited to those commercially available and commonly used in the construction industry in Australia especially high strength steel to AS1397. Further investigation on different types of bolts and steel materials is suggested to provide larger datasets.

Although various types of configurations of the conventional bolted connections undergoing curling effect were conducted in the experiments (i.e., 1 hole, 2 holes and 4 holes), further testing on different staggered hole configurations undergoing the presence of the curling phenomenon is required in order to fully confirm the effectiveness and accuracy of the proposed design equations and models for the combined effects of curling phenomenon and inclined shear.

In the shear beam-end connection tests, the cold-formed steel channels were connected to the T-shaped sections using bolts in order to be subjected to a combination of shear and rotation. The studied parameters of this experimental program can be expanded by using different thicknesses and strengths of the cold-formed steel channels as well as varying the number and distance of bolts used to investigate the change in initial stiffness, rotational rigidity, and group effect of the connections. Moreover, the shear beam-end connections were carried out by using the conventional bolted connections. Further testing on staggered hole configurations in conjunction with the curling effect are suggested to further confirm the effectiveness and accuracy of the proposed design equations and models for the combined effects of curling phenomenon, inclined shear and shear-rotation. Last but not least, the potential applications of bolted connections can also be studied by using an additional actuator to impose an axial force in the tested members and connections.

In conclusion, the thesis has made significant progress in researching the fracture behaviour and shear capacity of bolted connections in structural applications in high strength CFS sections. However, there is considerable scope to expand the range of this topic in the future for different configurations and steels.

NOTATION LIST

The following symbols are used in this paper:

A_{av} = active shear area lying in the middle of the net and gross shear areas;

A_{av}^{rs} = active shear area based on the resistant length (e_{rs});

A_b = gross cross-sectional area of the bolt;

A_c = minor diameter area of a bolt;

A_{gv} = gross area subject to shear;

A_{nt} = net area subject to tension;

A_{nv} = net area subject to shear;

A_o = plain shank area of a bolt;

A_s = tensile stress area of bolt;

b = curled length;

C = bearing factor;

c_d = dilatational wave speed of the steel material;

C_p = correction factor;

D = length between the two stagger holes;

d_f = bolt diameter;

d_h = diameter of the hole;

D_m = mean value of dead load;

E = Young's modulus;

e_1 = end distance;

e_2 = edge distance;

e_{net} = clear distance between end of material and edge of fastener hole or weld;

e_{rs} = resistant length;

f_{cr} = plate elastic buckling stress;

F_m = mean ratio of the actual thickness of the steel section to the specified value;

f_n = nominal strength of bolt;

f_{nt} = nominal tensile strength of bolt;

f_{nv} = nominal shear strength of bolt;

f_u = tensile strength of the connected parts;

f_{uf} = minimum tensile strength of bolt;

f_y = yield stress of the connected parts;

G = shear modulus;

g = gauge;

h_{wc} = coped flat web depth;

k = buckling coefficient;

k_t = correction factor for distribution of forces;

L = actual centerline distance;

L' = length of the failure path;

L_e = smallest characteristic length of the elements;

L_m = mean value of live load;

m_f = modification factor for type of bearing connection;

n = number of gauges g ;

n_b = number of fasteners along failure path;

N_f = design tensile capacity on the net section;

N_{ft} = nominal tensile capacity of bolt;

n_n = number of shear planes with threads intercepting the shear plane;

n_s = number of bolts in the considered section;

n_T = number of tests;

n_x = number of shear planes without threads intercepting the shear plane;

P^{FE} = Finite Element result;

P_1^T = Test result of the first test;

P_2^T = Test result of the second test;

P^T = Average test result;

P_m = mean ratio of the experimentally determined shear strength to the predicted shear strength;

P_n = nominal bolt strength;

P_{nt} = nominal tensile rupture strength;

Q_i = load effect;

Q_m = mean load effect;

R_d = design capacity;

R_m = given mean resistance;

R_n = nominal block shear strength;

R_{nt} = nominal pure tension rupture strength;

R_{nv} = nominal pure shear rupture strength;

R_n^S = shear strength on the failure path parallel to the staggered path;

R_n^T = tension strength on the failure path perpendicular to the staggered path;

R_u = nominal capacity;

R_u^S = shear fracture strength;

R_y^S = shear yielding strength;

R^T = tensile fracture strength;

s = shear distance between bolts;

s' = stagger;

s_f = bolt spacing in the tensile resistance plane;

t = thickness of steel sheet;

U_{bs} = nonuniform block shear factor;

U_{sl} = shear lag factor;

V_b = bearing capacity;

V_D = coefficient of variation of D_m ;

V_F = coefficient of variation of F_m ;

V_f = nominal tear-out strength;

V_{fv} = nominal shear capacity of bolt;

V_L = coefficient of variation of L_m ;

V_M = coefficient of variation of M_m ;

V_P = coefficient of variation of P_m ;

V_Q = coefficient of variation of Q_m ;

V_R = coefficient of variation of R_m ;

w = nominal width of steel sheet;

α = angle between the stagger and line of action of the force;

β = reliability index;

σ_{nom} = nominal stress;

σ_{true} = true stress;

ε_{nom} = nominal strain;

ε_{true} = true strain;

Δ_t = stable time increment;

ρ = effective width factor;

ν = Poisson's ratio;

Ω = safety factor;

λ = slenderness ratio;

π = pi;

γ_i = load factor; and

ϕ = resistance factor.

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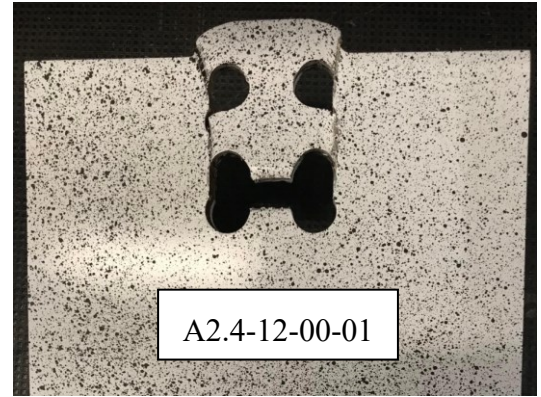
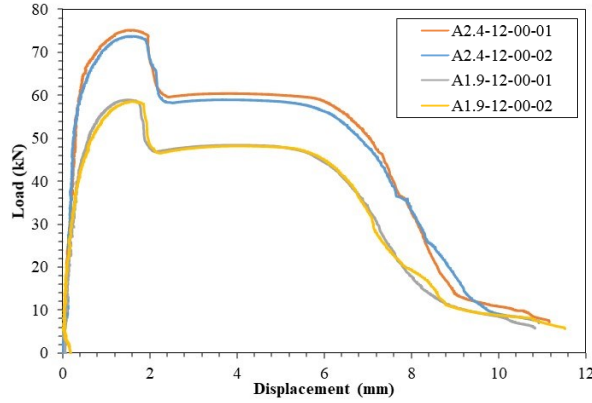
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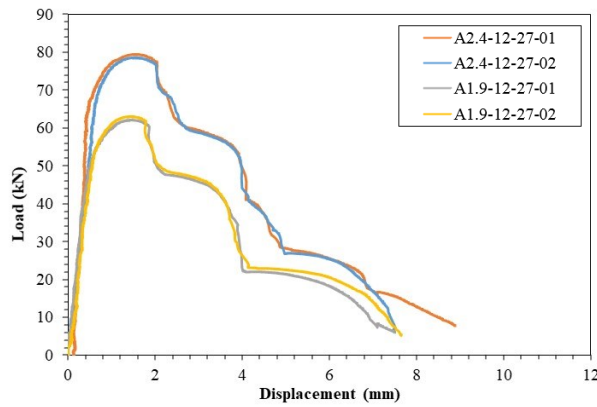
APPENDIX A

STAGGERED HOLE BOLTED CONNECTIONS TEST RESULTS

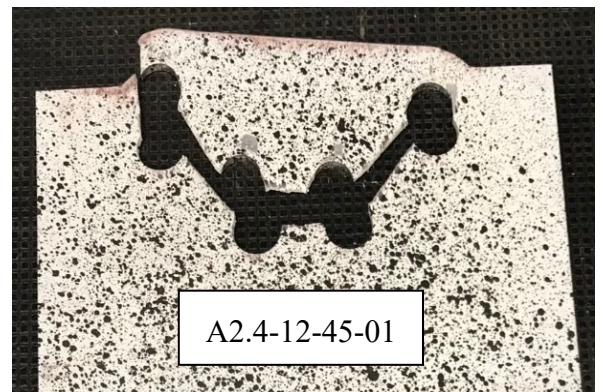
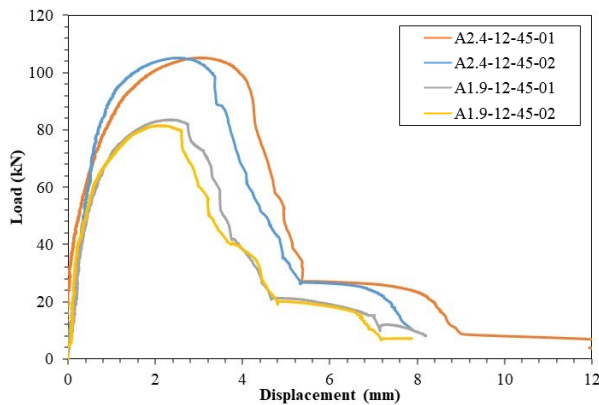
Load-displacement graphs and fracture behaviour of the staggered hole bolted connections



(a)

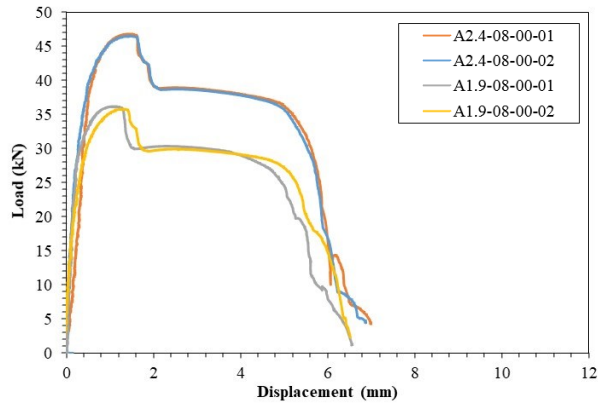


(b)

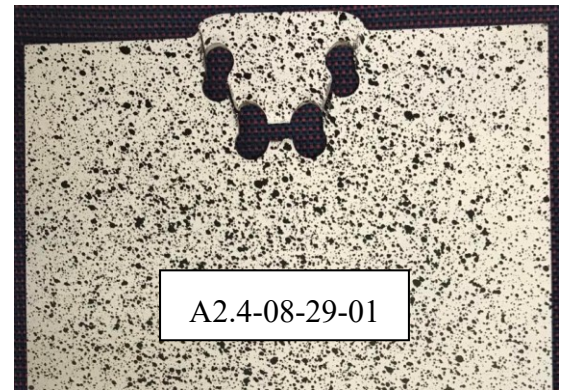
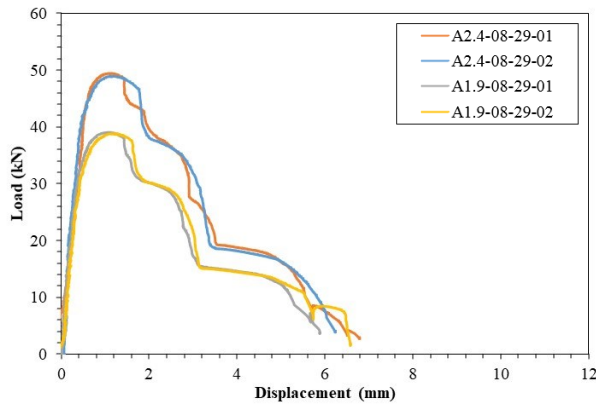


(c)

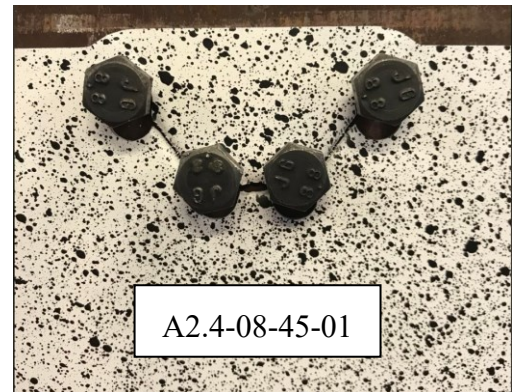
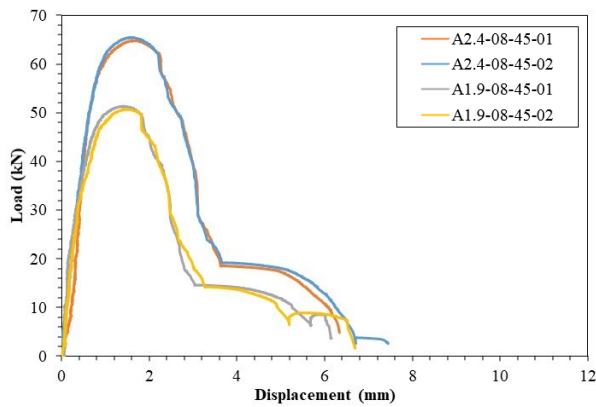
Figure A.1: Test results of the cold-formed steel staggered hole bolted connections using M12-Bolts (a) with the angle $\alpha = 0^\circ$, (b) with the angle $\alpha = 27^\circ$ and (c) with the angle $\alpha = 45^\circ$



(a)



(b)



(c)

Figure A.2: Test results of the cold-formed steel staggered hole bolted connections using M8-Bolts (a) with the angle $\alpha = 0^\circ$, (b) with the angle $\alpha = 29^\circ$ and (c) with the angle $\alpha = 45^\circ$

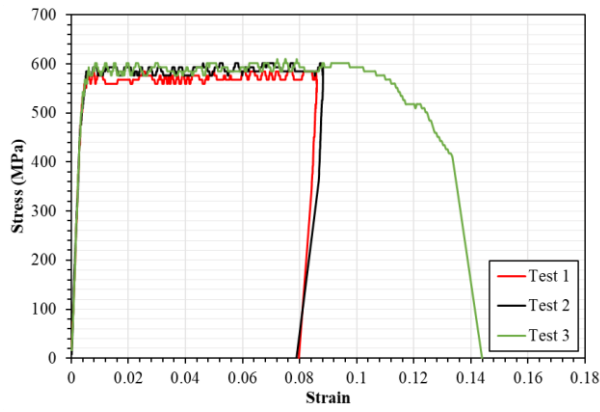
APPENDIX B

COUPON TEST RESULTS

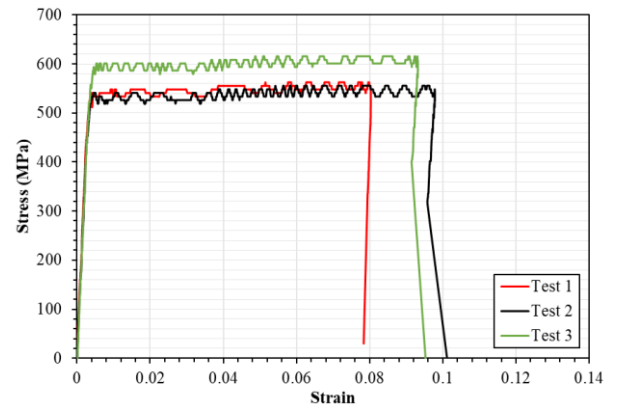
Coupon specimens were cut from cold-formed sheet steels used in the experiments. The specimens were taken in the longitudinal and transverse directions using a waterjet cutting machine. Details of the coupon thicknesses and properties are given as follows

Table B.1: Test results of the cold-formed sheet steels under tensile load

Specimen	Grade of steel	Direction	Nominal thickness t (mm)	Base metal measured thickness (mm)	Yield stress f_y (MPa)	Ultimate strength f_u (MPa)	Elongation (%)
T1.0-HG-01	G550	Longitudinal	1.00	0.99	559.31	584.36	-
T1.0-HG-02	G550	Longitudinal	1.00	0.99	559.31	601.05	-
T1.0-HG-03	G550	Longitudinal	1.00	0.99	567.66	601.06	13.3
T1.2-HG-01	G500	Longitudinal	1.20	1.18	533.43	562.66	-
T1.2-HG-02	G500	Longitudinal	1.20	1.18	579.71	615.94	9.9
T1.2-HG-03	G500	Longitudinal	1.20	1.18	540.74	555.35	-
T1.5-HG-01	G450	Longitudinal	1.50	1.48	579.40	616.61	-
T1.5-HG-02	G450	Longitudinal	1.50	1.48	584.72	616.61	14.2
T1.5-HG-03	G450	Longitudinal	1.50	1.48	579.40	616.61	-
T1.9-HG-01	G450	Longitudinal	1.90	1.89	545.69	576.25	12.2
T1.9-HG-02	G450	Longitudinal	1.90	1.89	550.06	571.88	-
T1.9-HG-03	G450	Longitudinal	1.90	1.89	532.59	560.97	-
T2.4-HG-01	G450	Longitudinal	2.40	2.37	521.65	547.79	11.2
T2.4-HG-02	G450	Longitudinal	2.40	2.37	516.12	543.24	-
T2.4-HG-03	G450	Transverse	2.40	2.37	588.90	605.91	-
T2.4-HG-04	G450	Transverse	2.40	2.37	589.95	606.74	12.6
T1.9-LG-05	G300	Longitudinal	1.90	1.89	299.21	362.50	-
T1.9-LG-02	G300	Longitudinal	1.90	1.89	295.14	362.31	-
T1.9-LG-03	G300	Longitudinal	1.90	1.89	296.23	362.21	62.4

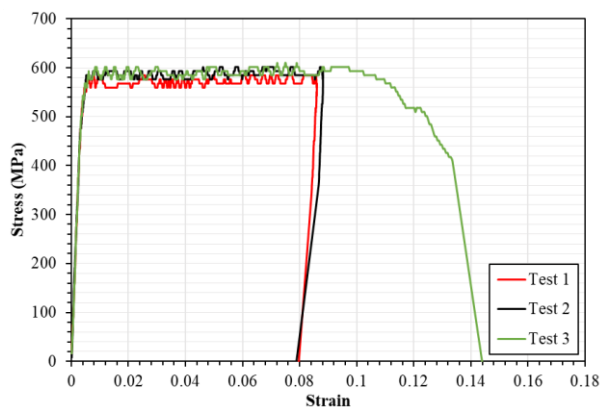


(a)

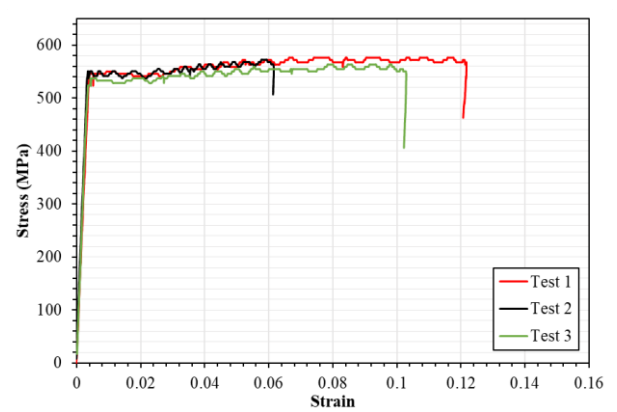


(b)

Figure B.1: Test results of (a) G550 1.0mm steel and (b) G500 1.2mm steel

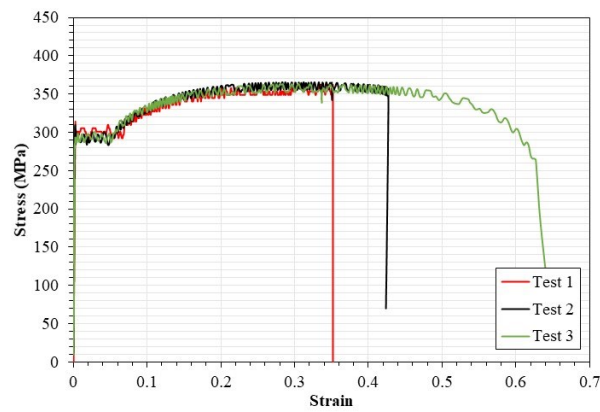


(a)

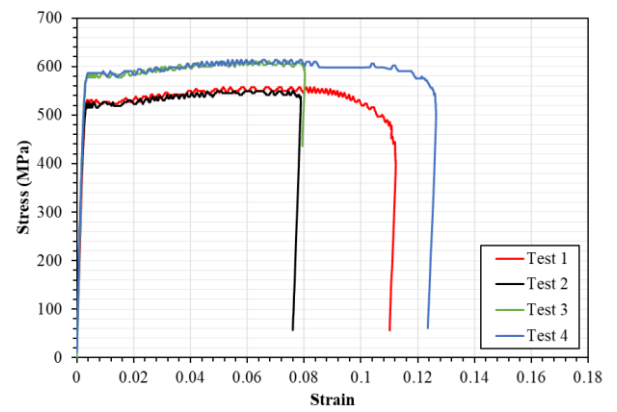


(b)

Figure B.2: Test results of (a) G450 1.5mm steel and (b) G450 1.9mm steel



(a)



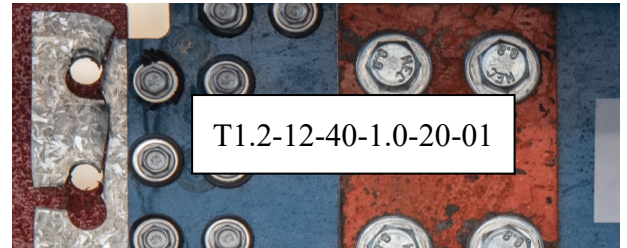
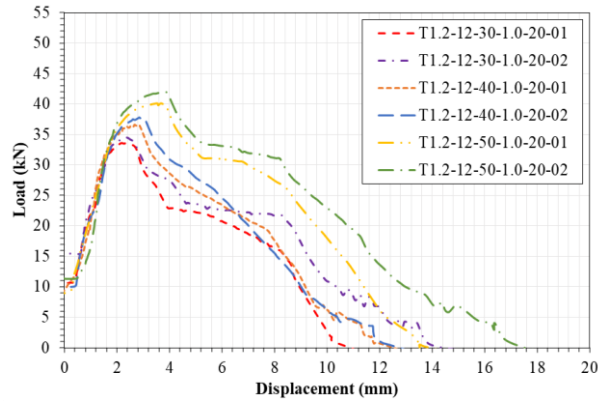
(b)

Figure B.3: Test results of (a) G300 1.9mm steel and (b) G450 2.4mm steel

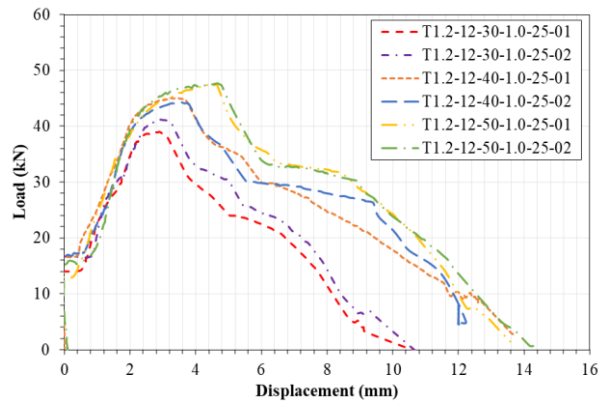
APPENDIX C

HIGH STRENGTH SHEAR BEAM-END BOLTED CONNECTIONS IN COLD-FORMED STEEL CHANNELS RESULTS

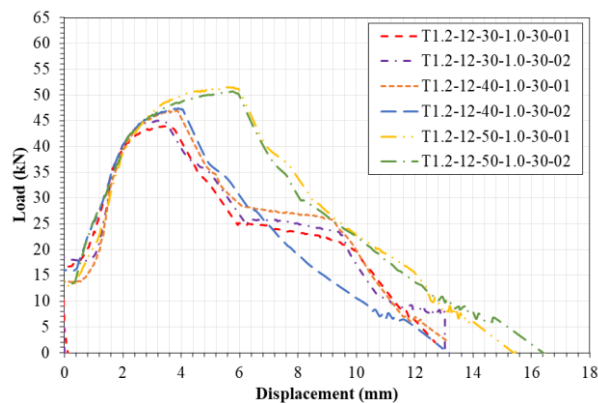
Load-displacement graphs and fracture behaviour of the shear beam-end bolted connections



(a)

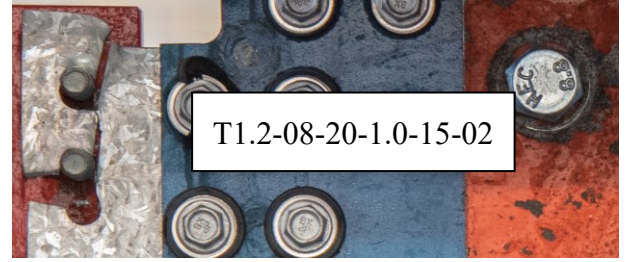
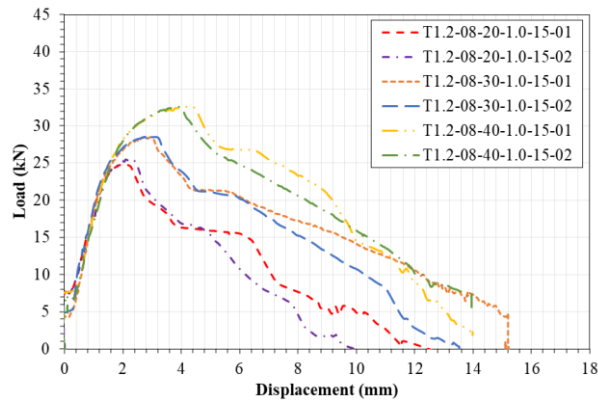


(b)

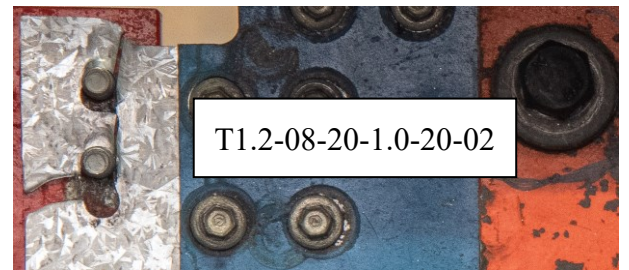
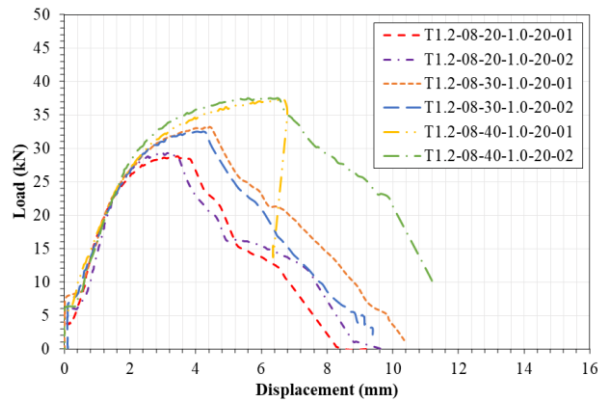


(c)

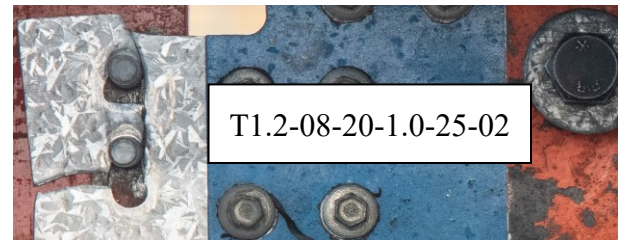
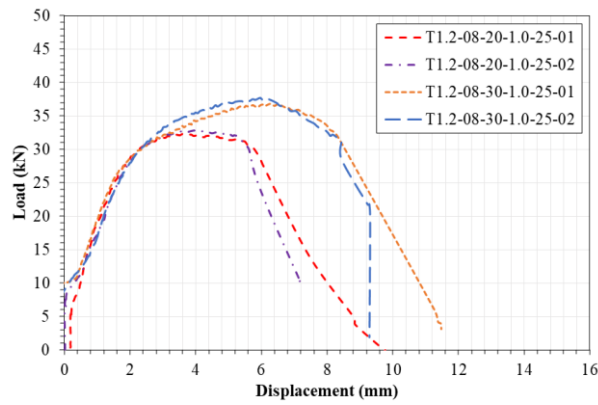
Figure C.1: Test results of the 1.2mm CFS bolted connections using M12-bolts (RR=1.0) with (a) $e_2 = 20\text{mm}$, (b) $e_2 = 25\text{mm}$ and (c) $e_2 = 30\text{mm}$



(a)

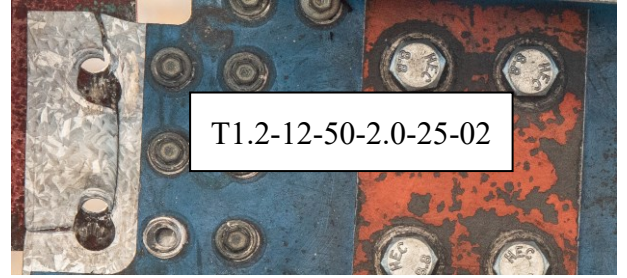
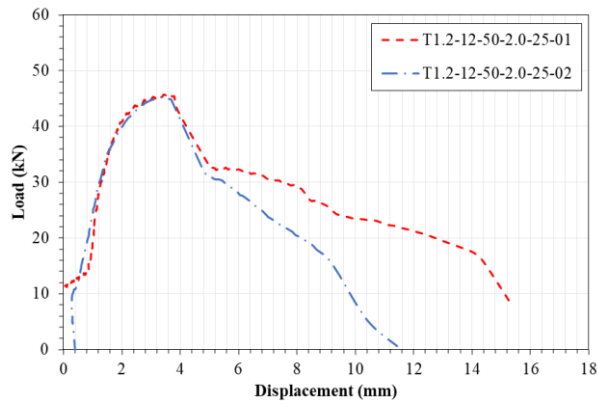


(b)

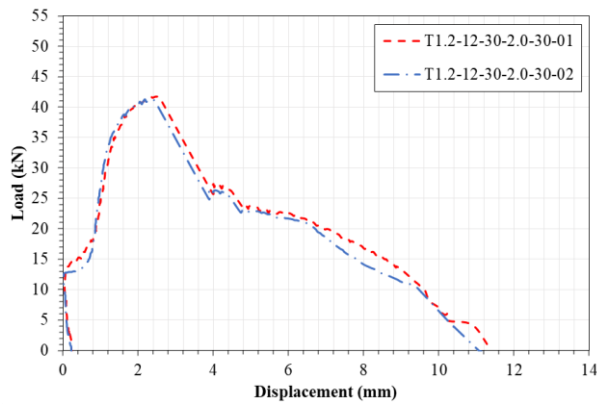


(c)

Figure C.2: Test results of the 1.2mm CFS bolted connections using M8-bolts (RR=1.0) with (a) $e_2 = 15mm$, (b) $e_2 = 20mm$ and (c) $e_2 = 25mm$



(a)



(b)

Figure C.3: Test results of the 1.2mm CFS bolted connections using M12-bolts (RR=2.0) with (a) $e_2 = 25mm$ and (b) $e_2 = 30mm$

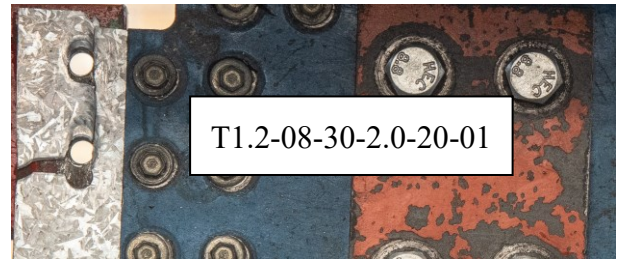
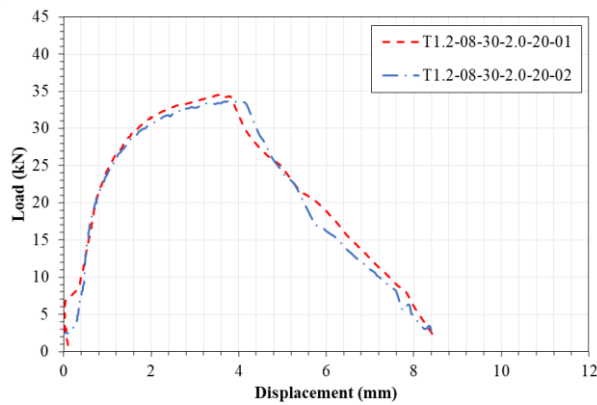
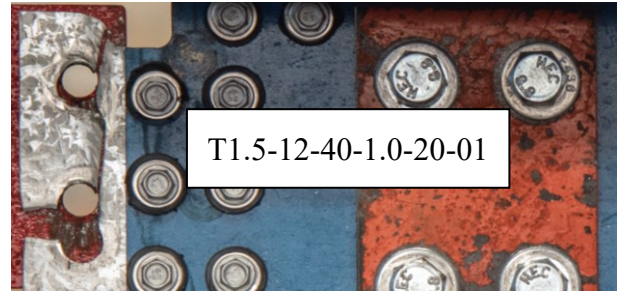
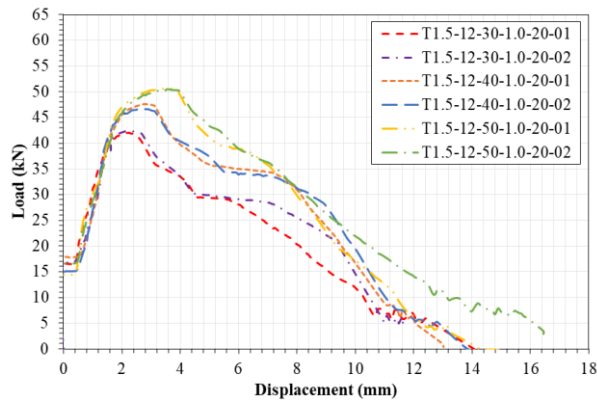
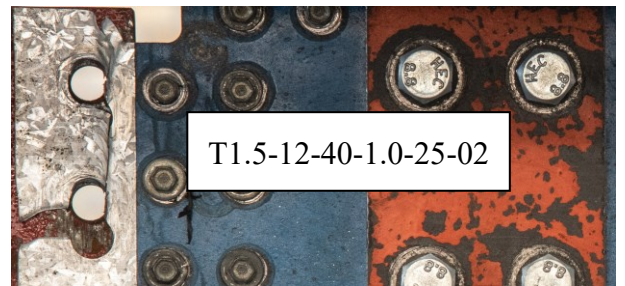
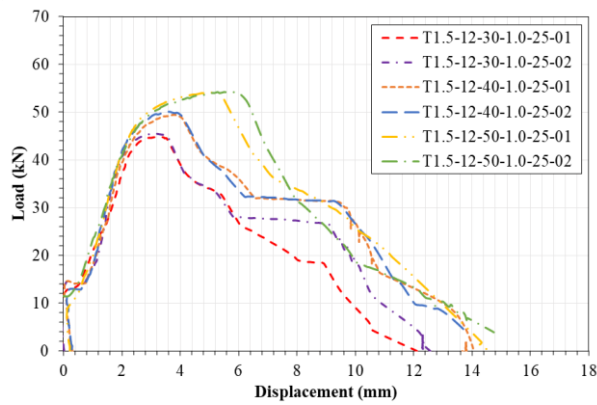


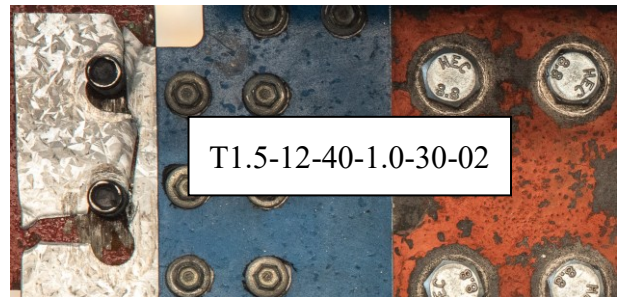
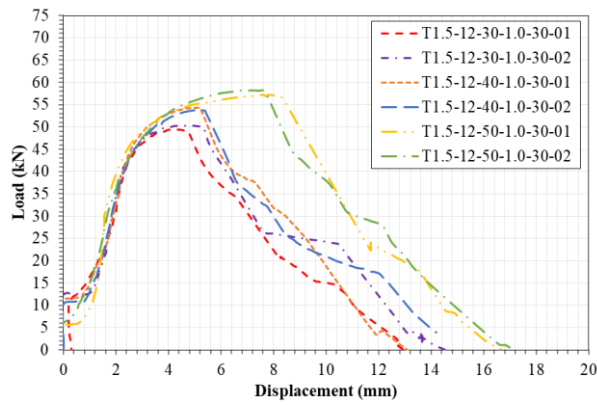
Figure C.4: Test results of the 1.2mm CFS bolted connections using M8-bolts (RR=2.0) with $e_2 = 20mm$



(a)

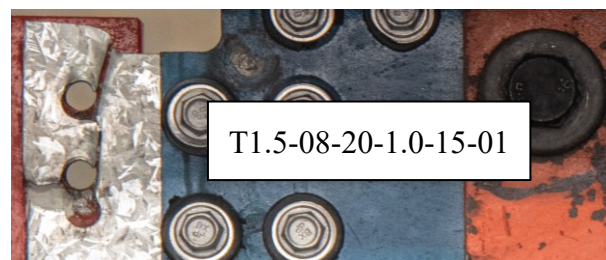
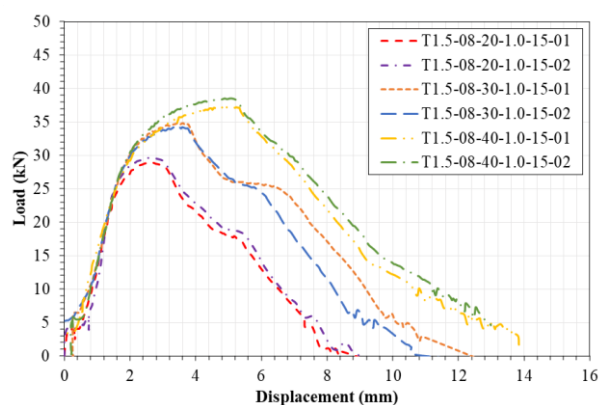


(b)

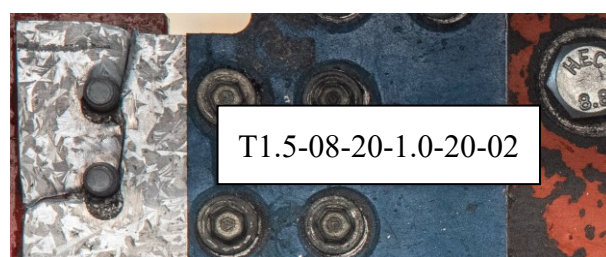
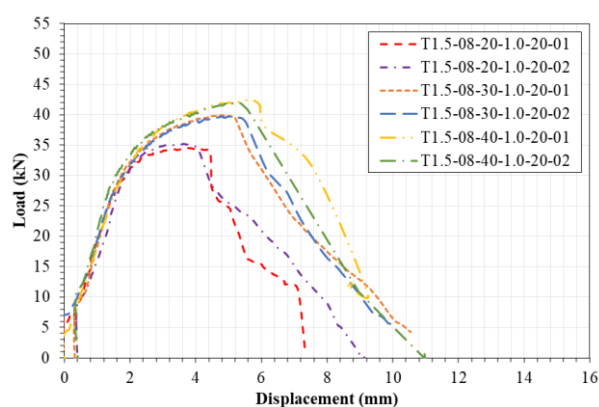


(c)

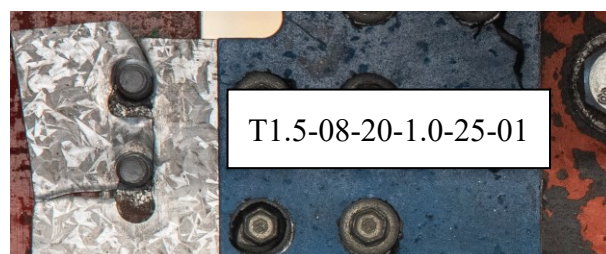
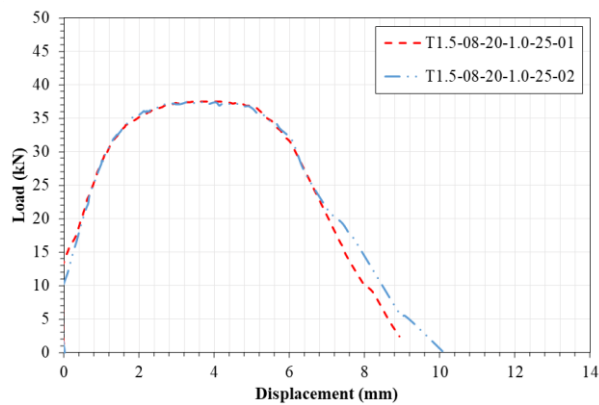
Figure C.5: Test results of the 1.5mm CFS bolted connections using M12-bolts (RR=1.0) with (a) $e_2 = 20\text{mm}$, (b) $e_2 = 25\text{mm}$ and (c) $e_2 = 30\text{mm}$



(a)



(b)



(c)

Figure C.6: Test results of the 1.5mm CFS bolted connections using M8-bolts (RR=1.0) with (a) $e_2 = 15mm$, (b) $e_2 = 20mm$ and (c) $e_2 = 25mm$

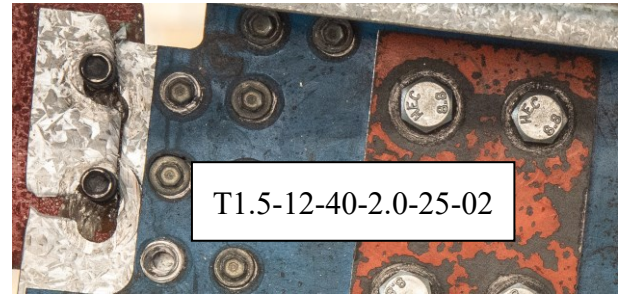
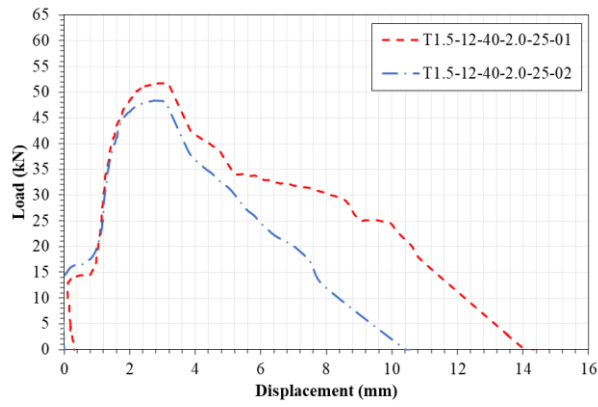


Figure C.7: Test results of the 1.5mm CFS bolted connections using M12-bolts (RR=2.0) with $e_2 = 25mm$

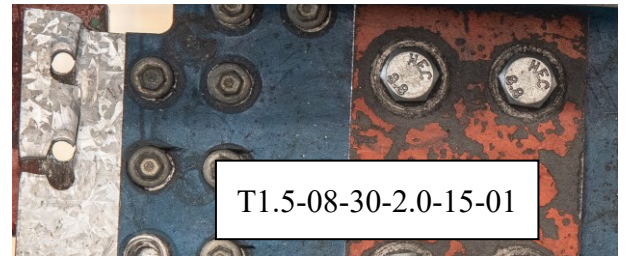
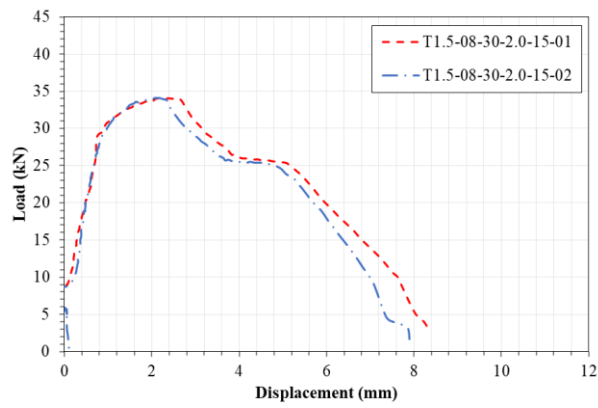
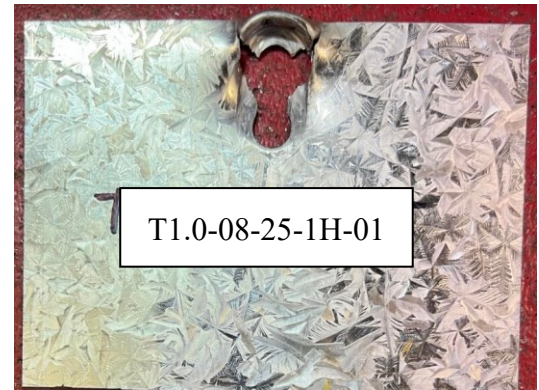
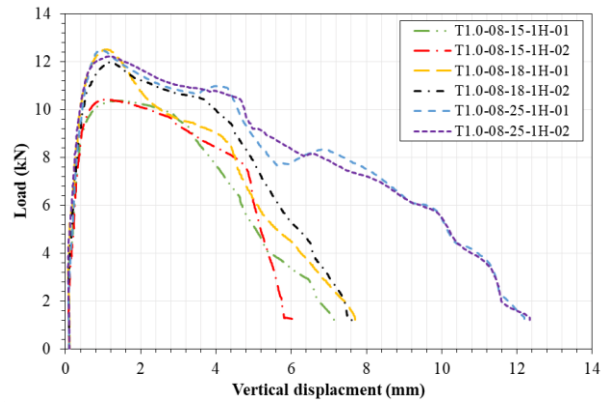


Figure C.8: Test results of the 1.5mm CFS bolted connections using M8-bolts (RR=1.0) with $e_2 = 15mm$

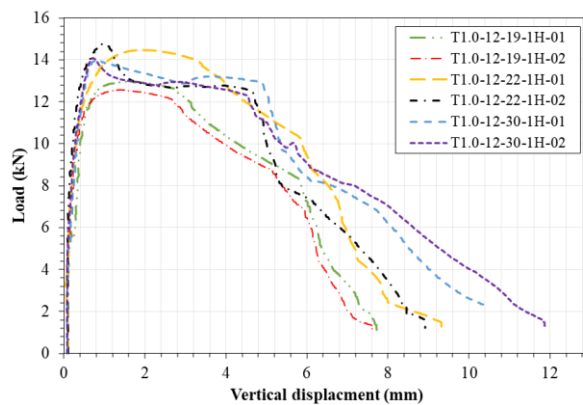
APPENDIX D

BOLTED CONNECTIONS UNDER CURLING EFFECT TEST RESULTS

Load-displacement graphs and fracture behaviour of the bolted connections under curling effect

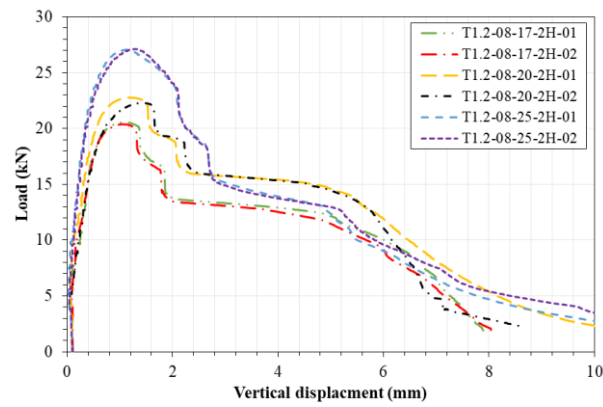


(a)

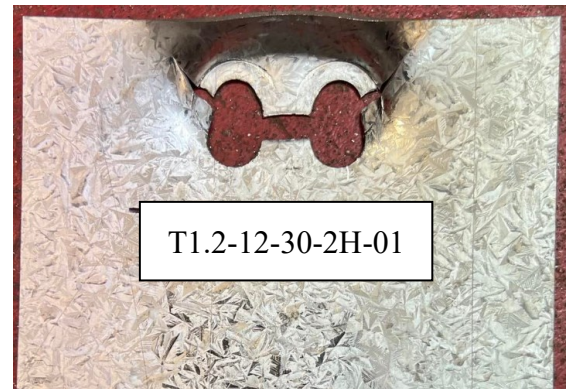
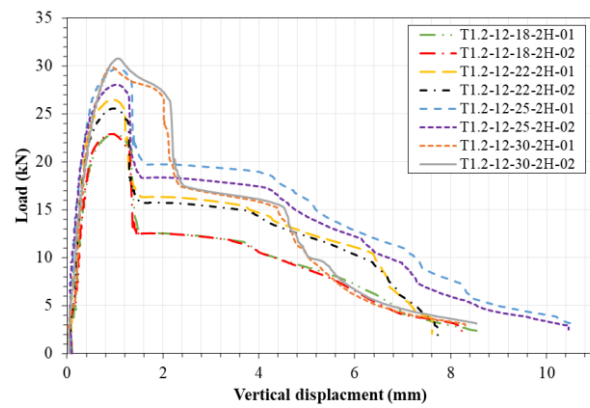


(b)

Figure D.1: Test results of the 1.0 mm G550 cold-formed steel bolted connections Configuration Type 1 using (a) M8-bolts and (b) M12-bolts

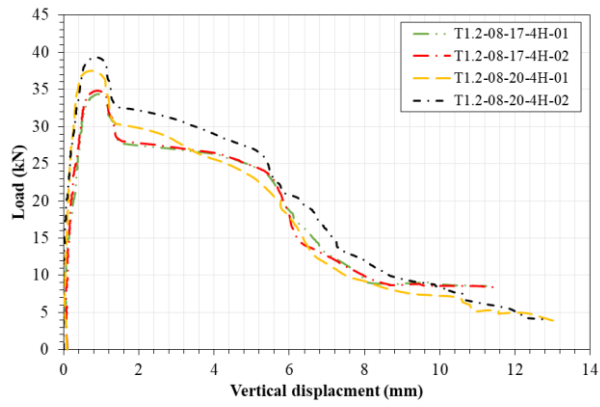


(a)

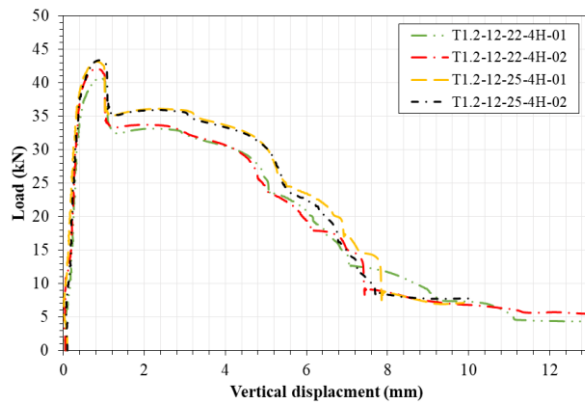


(b)

Figure D.2: Test results of the 1.2 mm G500 cold-formed steel bolted connections Configuration Type 2 using (a) M8-bolts and (b) M12-bolts

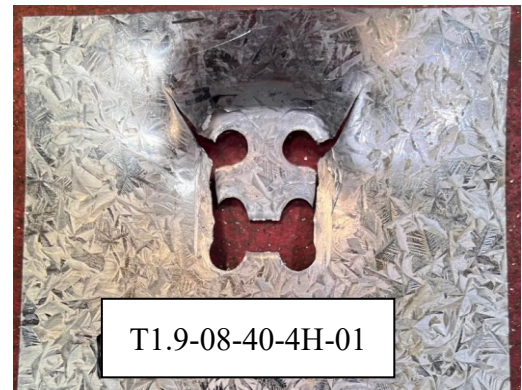
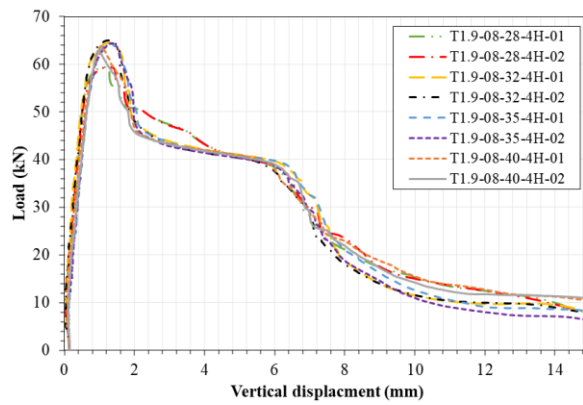


(a)

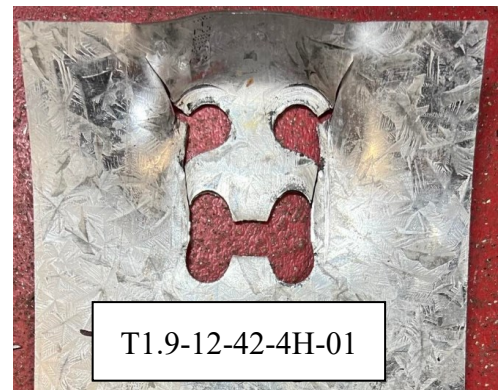
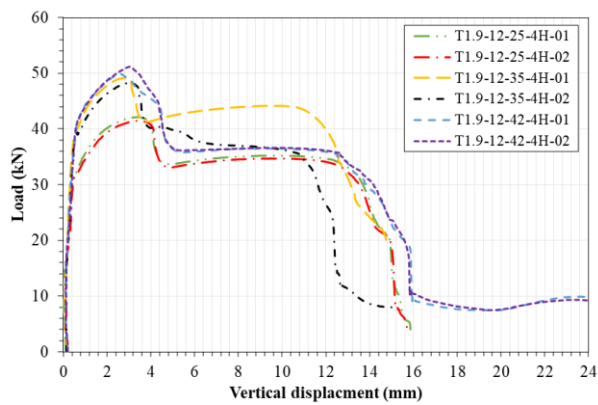


(b)

Figure D.3: Test results of the 1.2 mm G500 cold-formed steel bolted connections Configuration Type 3 using (a) M8-bolts and (b) M12-bolts



(a)



(b)

Figure D.4: Test results of the 1.9 mm (a) G450 cold-formed steel bolted connections Configuration Type 3 using M8-bolts and (b) G300 cold-formed steel bolted connections Configuration Type 3 using M12-bolts

APPENDIX E

FE MODEL RESULTS

Figure E.1-Figure E.4 illustrate the FE results of four different types of configurations (i.e., Configuration Type A, B, C and D) of the staggered hole bolted connections

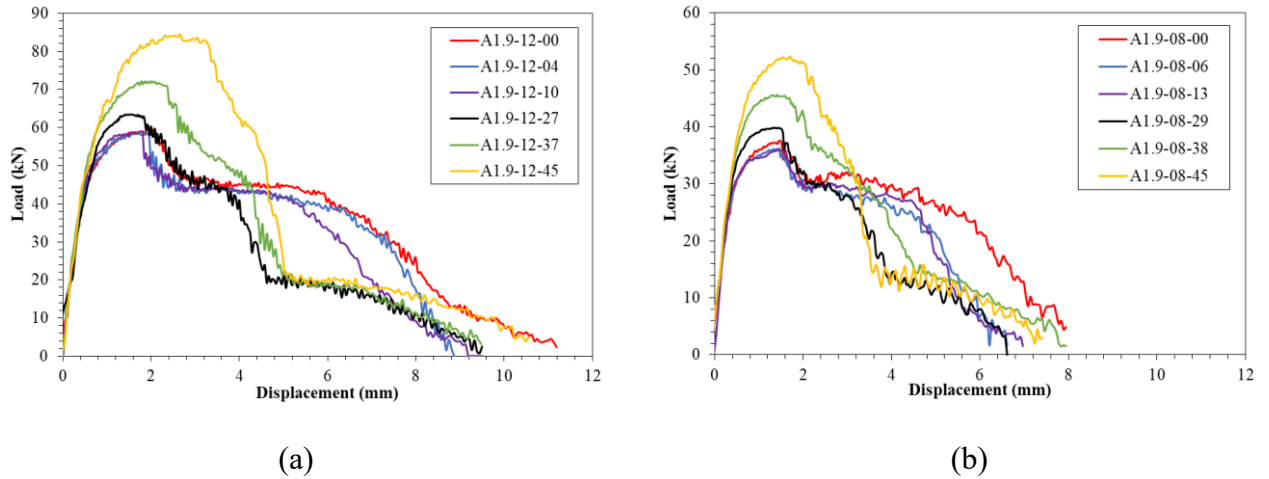


Figure E.1: FE results of 1.9mm cold-formed steel staggered hole bolted connections Configuration Type A using (a) M12-Bolts and (b) M8-Bolts.

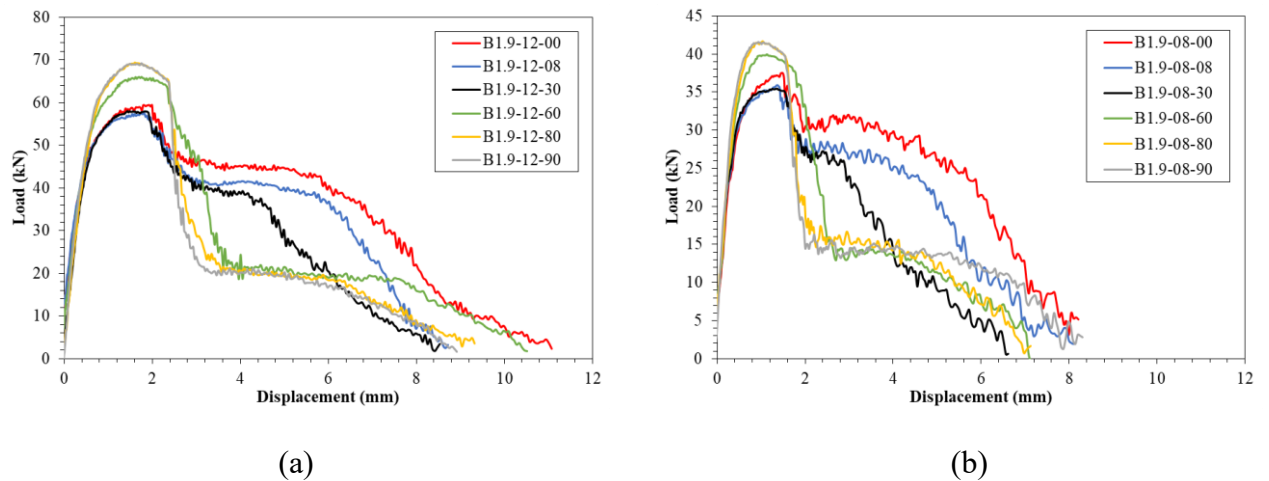
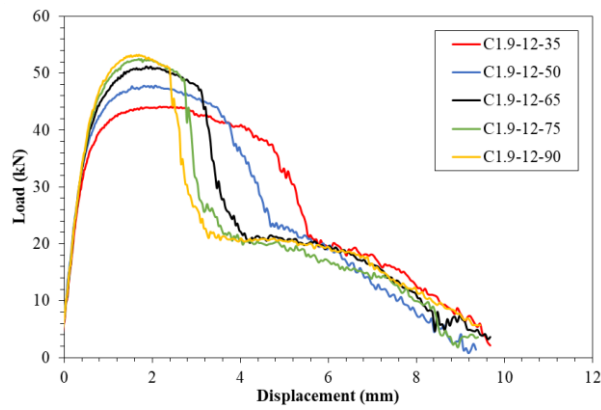
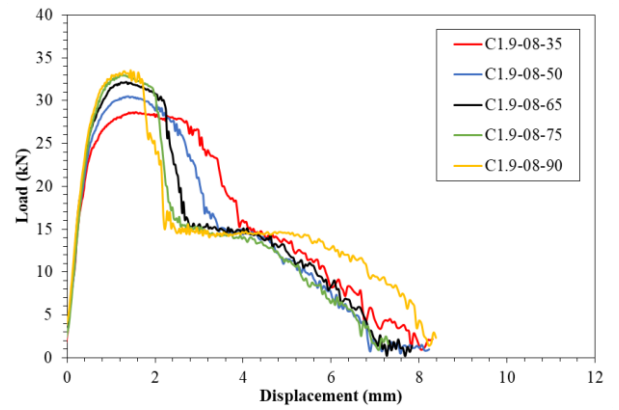


Figure E.2: FE results of 1.9mm cold-formed steel staggered hole bolted connections Configuration Type B using (a) M12-Bolts and (b) M8-Bolts.

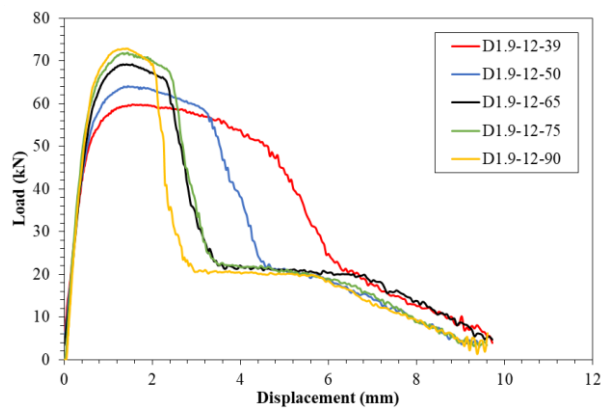


(a)

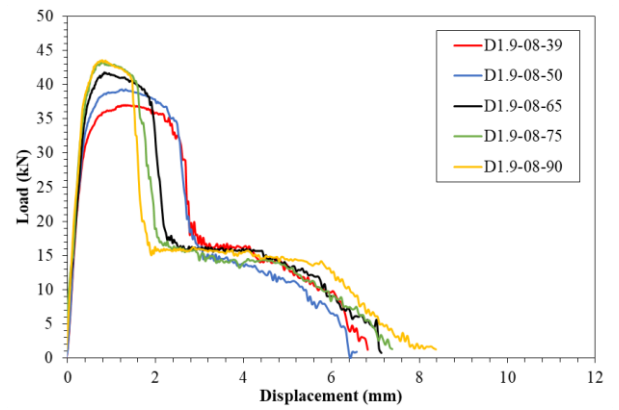


(b)

Figure E.3: FE results of 1.9mm cold-formed steel staggered hole bolted connections Configuration Type C using (a) M12-Bolts and (b) M8-Bolts.



(a)



(b)

Figure E.4: FE results of 1.9mm cold-formed steel staggered hole bolted connections Configuration Type D using (a) M12-Bolts and (b) M8-Bolts.