FIRE PERFORMANCE OF HYBRID TIMBER CONNECTIONS

by

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Abstract

Wood is being widely used as a construction material; however, the use of wood has been restricted by building codes because it is combustible. Hybrid building systems involve the use of heavy timber in combination with other materials. A hybrid timber connection is considered as the weakest link in this construction system particularly under fire exposure. Analysis of hybrid timber connections is complicated because the behavior and properties of the different materials are different under fire exposure.

This study aims at investigating the effect of different parameters on the fire performance of hybrid connections between a glulam beam and a steel column by conducting full-scale tests and finite element modeling. The experimental part of this research consisted of a series of full-scale tests conducted in compliance with CAN/ULC-S101. Three different types of connections were investigated, the Concealed shear tabs, the Exposed shear tabs with 2 and 4 bolts and the Seated connections with one bolt and without a bolt. Two sizes of bolts 12.7 mm and 19.1 mm were used and each connection was tested for three load ratios 30%, 60% and 100%.

Test results showed that the Seated connection without a bolt outperformed the Exposed and the Concealed connections. It was noticed that at 100% load ratio the failure times for all connections were less than 40 minutes. Lowering the load ratio from 100% to 30% increased the failure time by 50 to 72%. Increasing the bolt size reduced the failure time by 20 to 48 %. Reducing the number of bolts from 4 to 2 increased the failure time by 10 to 86% and removing the bolt from the Seated connection increased the failure time by 50 to 125%. The final failure mode was always brittle / splitting of wood, even

Fire Performance of Hybrid Timber Connections

when ductile failure (yielding of bolts or crushing of wood) was the governing mode in ambient design. Three empirical formulas are proposed by the author to calculate the failure time for design purposes for the three different connections.

Dedication

To my wife Zenah for her invaluable support without which I would not be able to complete this study, to my beloved children Noorhan, Ali and Aws and to my family and my wife's family back home for their continuous support in good and bad times.

I love you all

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Table of Contents

Abstract.....	i
<i>Dedication</i>.....	iii
Acknowledgements	iv
Table of Contents	v
List of Tables	ix
List of Figures.....	x
1 Introduction.....	1
1.1 Problem Statement	1
1.2 Previous Work in this Field	4
1.3 Scope and Objectives	7
1.4 Methodology	8
2 Literature Review	9
2.1 Fire Safety.....	9
2.2 Behavior of Wood and Steel in Timber Connections	11
2.2.1 Behavior at Ambient Temperature.....	11
2.2.2 Potential Failure Modes	15
2.2.3 Hybrid Connections at Ambient Conditions.....	18
2.2.4 Hybrid Connections in Fire.....	30
2.2.5 Numerical Modelling.....	33
2.3 Wood Properties at Elevated Temperature	34
2.3.1 Wood Thermal Conductivity	35
2.3.2 Specific Heat.....	36
2.3.3 Density	37
2.3.4 Modulus of Elasticity.....	38
2.3.5 Tensile Strength	39

2.3.6	Compressive Strength	40
2.3.7	Shear Modulus	41
2.3.8	Shear Strength.....	41
2.4	Steel Properties at Elevated Temperature	42
2.4.1	Steel Thermal Conductivity	42
2.4.2	Steel Specific Heat.....	43
2.4.3	Density	44
2.4.4	Reduction Factor for Steel Mechanical Properties	44
2.4.5	Eurocode Equations for Mechanical Properties.....	45
3	Experimental Research	47
3.1	The Furnace	47
3.2	Assembly Temperature	49
3.3	Furnace Temperature Verification	54
3.4	Description of Experiments and Test Specimens	56
3.4.1	Concealed Steel Plate.....	59
3.4.2	Exposed Steel Plate.....	60
3.4.3	Seated Bearing plate	61
3.5	Test Procedure	61
3.6	Load Level	62
3.7	Test Plan.....	63
4	Finite Element Modeling	65
4.1	Heat Transfer Model	65
4.2	Heat Transfer Between Members	66
4.3	Structural Model	67
4.4	Damage and Failure Model.....	68
4.5	Discretization	69
4.6	Assembly and Contact Interactions	72
4.7	Loads and Boundary Conditions.....	73
4.8	Analysis.....	76
4.9	Finite Element Model Simulations	79

5	Results and Discussion.....	82
5.1	Fire Performance Tests for the Concealed Plate Connections with 4 Bolts	
	Group 1 and Group 2	82
5.1.1	Furnace Temperature	82
5.1.2	Deflection.....	87
5.1.3	Load Ratio.....	96
5.1.4	Specimen Temperature Profile	101
5.1.5	Charring Rate	132
5.1.6	Failure Time and Failure Mode	136
5.1.7	Model Validation	141
5.1.8	Discussion	153
5.2	Tests with Exposed Plate Connections with 4 Bolts Group 3 and Group 4 ...	157
5.2.1	Furnace Temperature	159
5.2.2	Deflection.....	160
5.2.3	Load Ratio.....	166
5.2.4	Specimen Temperature Profile	168
5.2.5	Charring Rate	181
5.2.6	Failure time and Failure Mode.....	182
5.2.7	Model validation	186
5.2.8	Discussion	196
5.3	Tests of the Seated Connections for Group 5	202
5.3.1	Deflection.....	202
5.3.2	Failure Time and Failure Mode	204
5.3.3	Model Validation	210
5.3.4	Discussion	216
5.4	Tests of the Exposed and the Concealed Connections with Two Bolts - Group 6	
	219	
5.4.1	Deflection.....	220
5.4.2	Failure Time and Failure Mode	222
5.4.3	Model Validation	222

Fire Performance of Hybrid Timber Connections

5.4.4	Discussion	227
5.5	Comparison of Failure Time of the Different Connections	231
5.5.1	Effect of the Number of Bolts.....	231
5.5.2	Failure Times for the Different Load Ratios.....	234
5.5.3	Summary	240
5.6	Empirical Relationship.....	241
5.6.1	Tests Data.....	242
5.6.2	Correlations for the CN and the EX Connections.....	243
5.6.3	Correlations for SE Connections	247
5.6.4	Correlation limitation.....	253
5.6.5	Summary	253
6	Summary, Conclusions and Recommendation.....	255
6.1	Summary	255
6.2	Conclusions.....	257
6.3	Recommendations for Future Work.....	260
	Appendix.....	262
	References.....	267

List of Tables

Table 1- Experimental Tests Groups	64
Table 2 – Finite element modeling tests	81
Table 3 – The charring rates for Group 1 and 2.....	135
Table 4 - Time and Failure mode for Group 1 & 2.....	140
Table 5 - Model test for Groups 1 and 2.....	142
Table 6- Test and model time of failure.....	152
Table 7 -Table 3 - Charring rates for Group 3 and 4	181
Table 8 - Time and Failure mode for Group 3 & 4.....	185
Table 9 -Model test for Group 3 and 4	186
Table 10- The test and model times of failure for Groups 3 and 4.....	195
Table 11 -Time and Failure mode for Group 5.....	209
Table 12 - Model test for Groups 5 and 6.....	210
Table 13 -Test and model time of failure for the Seated connections, Groups 5 and 6..	215
Table 14 - Time and Failure mode for Group 6.....	223
Table 15 - Test and model time of failure for the 2 bolt connections in Groups 7, 8, 9 and 10.....	224
Table 16 - Test and model time of failure for 2 bolts connections for Group 6	226
Table 17 – Predicted failure times for the EX, the CN and the SE connections with a 62.5 kN load.....	239
Table 18 - Test data used to develop the empirical functions.....	243
Table 19 – Correlation and connection factors of the CN and EX connections.....	246
Table 20 - Correlation and connection factors M & N for the SE connections.....	252
Table 21 - Empirical formulas	254

List of Figures

Figure 1 - Standard time temperature curve	10
Figure 2 Three principal axes in wood (CSA O86) [36]	11
Figure 3- Stress Strain relationship for wood (Buchanan) [3].....	12
Figure 4 Typical failure modes of wood connections (CAN/CSA O86-09) [36].....	14
Figure 5- Ductile failure modes for double shear timber connections (CAN/CSA O86-09) [36].....	16
Figure 6- The GFRP - Concrete composite section: bolted (left) and bonded (right)- Joao [28].....	18
Figure 7 - Gustavo test frames [32]	20
Figure 8 - Gustavo test frames [32]	20
Figure 9 - Gustavo's proposed connection concept [32]	21
Figure 10 - Sudhakar typical connection details [29]	22
Figure 11 - Sudhakar's FE model at joint region [29].....	24
Figure 12 - R.A Hawileh's hybrid connection detail [31]	25
Figure 13 - R.A Hawileh's test structure [31].....	25
Figure 14 - R.A Hawileh test frame and FE model [31].....	26
Figure 15 -Seung FE mesh of the beam specimens [30]	27
Figure 16 – Seung's details of the beam specimen [30]	28
Figure 17 – Jorge's composite timber - concrete joint configuration [34]	29
Figure 18 – Ing's Timber - concrete composite beam [53]	30
Figure 19 – S. Schnabl descriptive geometric of two layer timber composite beam [56]	31
Figure 20 - Thermal conductivity of wood at elevated temperatures [26]	36
Figure 21-Specific heat of wood at elevated temperatures (Peng, 2010) [72].....	37
Figure 22 - Density of wood at elevated temperatures (Peng) [72].....	38
Figure 23 - Modulus of elastic of wood at elevated temperatures (Peng) [72].....	39
Figure 24 - Tensile strength of wood at elevated temperatures [26].	40
Figure 25 - The compressive strength of wood at elevated temperatures (Peng) [26].	41
Figure 26 - Steel thermal conductivity variations with temperature (Eurocode 3) [79] ...	43
Figure 27- Steel specific heat variation with temperature (Eurocode 3) [79].....	44

Figure 28 - Mechanical properties of steel at elevated temperatures (Eurocode 3) [79]..	45
Figure 29- Exterior photo of the furnace	48
Figure 30- Interior photo of one of the two propane burners	48
Figure 31 -Shielded thermocouples and plate thermometers in the furnace.....	48
Figure 32-Cross section of furnace with test assembly in place	50
Figure 33 - Cross section of steel beam and wood beam with the locations of the thermocouples.	51
Figure 34 - Cross section of furnace with LVDT	52
Figure 35- The locations of thermocouples and plate thermometer in the furnace	54
Figure 36 -Measured time-temperature curves and CAN/ULC-S101 standard curve without the wood specimen	55
Figure 37 - Measured time-temperature curves and CAN/ULC-S101 /ASTM E119 standard curve with a wood specimen	56
Figure 38 - Specimen placed in furnace	57
Figure 39 - Specimen in the furnace after the test	57
Figure 40 -Steel plate connection types; A: Concealed B: Exposed C: Seated	59
Figure 41 – CAN/CSA O86-09 bolt placement requirements in joints	60
Figure 42 -One quarter Model in ABAQUS	67
Figure 43 Damage Dissipated Energy (ALLDMD) of the whole model [10]	69
Figure 44 -Steel column meshed model.....	70
Figure 45- Glulam beam mesh model.....	70
Figure 46 - Plate mesh model	71
Figure 47 - Bolt mesh model	71
Figure 48 - Recorded temperature within the furnace	74
Figure 49 - Applied heat flux load on the assembly	75
Figure 50 - Mechanical boundary condition of model assembly.....	76
Figure 51 - Concealed specimen in the furnace before the test	83
Figure 52 - Furnace temperature vs. standard fire- CN 4-19.1-30%	83
Figure 53 – The furnace temperature vs. standard fire - CN 4-19.1-100%	85
Figure 54 – The furnace temperature vs. standard fire - CN 4-12.7.1 - 30%	85
Figure 55 – The furnace temperature vs. standard fire - CN 4-12.7 - 60%	86

Figure 56 – The furnace temperature vs. standard fire - CN 4-12.7-100%	86
Figure 57 - Beam deflection vs. time CN 4-19.1 - 30%	88
Figure 58 – The mid-span deflection vs. time CN 4-19.1 - 30%.....	89
Figure 59 - Beam deflection vs. time CN 4-19.1 - 100%	90
Figure 60 – The mid span deflection vs. time CN 4-19.1 - 100%	90
Figure 61 - Beam deflection vs. time CN 4-12.7- 30%	91
Figure 62 - Mid span deflection vs. time CN 4-12.7 - 30%.....	92
Figure 63 - Beam deflection vs. time CN 4-12.7 - 60%	93
Figure 64 – The mid-span deflection vs. time CN 4-12.7 - 60%.....	93
Figure 65 - Beam deflection vs. time CN 4-12.7 - 100%	94
Figure 66 - Mid span deflection vs. time CN 4-12.7 - 100%.....	95
Figure 67 – Load Ratio vs Time - CN-4-19.1- 30%	97
Figure 68 – Load Ratio vs Time - CN 4-19.1 - 100%	98
Figure 69 - Load Ratio vs Time - CN 4-12.7-30%	99
Figure 70 – Load Ratio vs Time - CN 4-12.7 - 60%	100
Figure 71 – Load Ratio vs Time - CN 4-12.7-100%	100
Figure 72 – The steel column's temperature vs. time -CN 4-19.1 - 30%.....	102
Figure 73 – The steel column's temperature vs. time -CN 4-19.1 - 100%.....	103
Figure 74 – The steel column's temperature vs. time -CN 4-12.7 - 30%.....	104
Figure 75 – The steel column's temperature vs. time -CN 4-12.7 - 60%.....	104
Figure 76 – The steel column's temperature vs. time -CN 4-12.7 - 100%.....	105
Figure 77 – The bolts & the plate temperature vs. time on the right side -CN 4-19.1- 30%	106
Figure 78 – The bolts and the plate temperature vs. time on the left side -CN 4-19.1 - 30%	107
Figure 79 – The steel plate and bolts for Test 1.1 -CN 4-19.1 - 30%.....	107
Figure 80 – The bolts and plate temperature vs. time on the left side -CN 4-19.1 - 100%	108
Figure 81 – The bolts and plate temperature vs. time on the right side CN 4-19.1 -100%	109
Figure 82 – The bolts for Test 1.2 CN 4-19.1 100%	109

Figure 83 – The bolts and plate temperature vs. time at right side -CN4-12.7 - 30%	111
Figure 84 – The bolts and plate temperature vs. time at the left side CN4-12.7 - 30%..	111
Figure 85 – The bolts for Test 2.1 CN 4-12.7 30%	112
Figure 86 – The bolts and plate temperature vs. time at the right side -CN4-12.7 - 60%	113
Figure 87 – The bolts and plate temperature vs. time at the left side-CN4-12.7 - 60% .	113
Figure 88 -Bolts for Test 2.2 -CN 4-12.7 - 60%.....	114
Figure 89 – The bolts and plate temperature vs. time on the right side -CN4-12.7 - 100%	115
Figure 90 – The bolts and plate temp vs. time on the left side CN4-12.7 - 100%.....	115
Figure 91 – The bolts for Test 2.3 CN 4-12.7 = 100%	116
Figure 92 – Wood section temperature vs time near the left connection -CN 4-19.1- 30%	118
Figure 93 – Wood section temperature vs time near the right connection -CN 4-19.1 - 30%	118
Figure 94 – The temp distribution along the beam’s width at right side connection - CN4- 19.1- 30%.....	120
Figure 95 - Temp Distribution along the beam’s width at connection - CN4-19.1 - 30%	120
Figure 96 – The wood’s temperature vs. time at the mid-section - CN 4-19.1-30%.....	121
Figure 97 – The temperature distribution along the beam’s depth vs. time CN 4-19.1 - 30%	122
Figure 98 –The temp distribution along the beam’s width at the mid span - CN 4-19.1 - 30%	123
Figure 99 – The wood’s temperature vs. time at the right side connection - CN 4-12.7- 30%	124
Figure 100 – The wood’s temp vs. time at the left side connections - CN 4-12.7 - 30%	124
Figure 101 – The wood’s temperature vs. time at the mid-span section - CN 4-12.7 - 30%	125
Figure 102 - Separation of the wood layer at the top of the beam & failure at the connection CN 4-12.7- 30%	125

Figure 103 – The temperature distribution along the beam’s width vs. time -CN 4-12.7 - 30%	126
Figure 104 –The temperature distribution along the beam’s depth vs. time -CN 4-12.7 - 30%	127
Figure 105 – The wood’s temperature vs. time at the left connection - CN 4-12.7 - 100%	128
Figure 106 – The wood’s temperature vs. time at the right connection - CN 4-12.7 - 100%	128
Figure 107 – The wood’s temperature vs. time at the mid-span section - CN 4-12.7 - 100%	129
Figure 108 – The temperature distribution along the beam’s depth vs. time CN 4-12.7 - 100%	130
Figure 109 – The temperature distribution along beam’s width vs. time at mid-span CN 4-12.7- 100%.....	130
Figure 110 – The temperature distribution along the beam’s width vs. time	131
Figure 111 – The temperature distribution along beam’s width vs. time at the left connection CN 4-12.7 - 100%	131
Figure 112- The residual width of the beam at the mid-span	134
Figure 113 – The residual depth of the beam at the mid-span.....	134
Figure 114 – The residual width at the connection.....	134
Figure 115 – The charring rates at the mid-span and connection for Groups 1 & 2	135
Figure 116 - Failure at the connection Test 1.1	136
Figure 117 - Failure at the connection; a) bolts and plates - Test 1.1; b) wood splitting - Test 1.2.....	137
Figure 118: a) Splitting of the wood - Test 2.1; b) Separation of the wood layers - Test 2.2.....	138
Figure 119 - Test 2.2 a) The separation in the wood layer; b) The separation in the wood layer (close shot)	138
Figure 120 – The slight bending of the bolts - Test 2.2	139
Figure 121 - Bolt temperature vs. time - T11	143
Figure 122 - Plate temperature vs. time - T12	144

Figure 123 – The wood section's temp vs. time - a) T16, b) T25, c) T28	146
Figure 124 - Temperature distribution in the model at 27 minutes	147
Figure 125 - Temperature distribution at the beam's width full scale test– Test 2.2	147
Figure 126 - Temperature distribution at the beam's depth – Test 2.2.....	148
Figure 127 - Temperature distribution due to the metal components at the Concealed connection	149
Figure 128 – The progressive damage of the Concealed connection [10] [86]	150
Figure 129 – The cracks generated at bolts' edge.....	151
Figure 130 - Strains at the bolts exceeded the steel's yield strain at 30.2 minutes.....	151
Figure 131 – Failure time of CN Connection; Test vs. Model	152
Figure 132 – Failure Time of the Concealed Connection for Groups 1 and 2.....	154
Figure 133 – The mid-span deflection vs. time Group 1	155
Figure 134 – The mid-span deflection vs. time Group 2	155
Figure 135 – The effect of the bolt diameter on the failure time for Groups 1 and 2.....	156
Figure 136- mid span deflection vs. time for Groups 1 and 2	157
Figure 137 – The Exposed connection specimen in the furnace before and after the test	158
Figure 138 - The furnace temperature vs time- EX 4-12.7-100% Test 3.2	159
Figure 139 – The furnace temperature vs time- EX 4-19.1-60% Test 4.1.....	160
Figure 140 –The beam deflection vs. time EX 4-12.7 60% - Test 3.1	161
Figure 141 The beam's mid-span deflection vs. time EX 4-12.7 60% - Test 3.1.....	161
Figure 142 – The beam deflection vs. time EX 4-12.7 100% - Test 3.2	162
Figure 143 – The beam's mid-span deflection vs. time EX 4-12.7 100% - Test 3.2.....	163
Figure 144 –The beam's mid-span deflection vs. time EX 4-19.1 60% - Test 4.1.....	163
Figure 145 – The beam's deflection vs. time EX 4-19.1 60% - Test 4.1	164
Figure 146 –The beam's mid-span deflection vs. time EX 4-19.1 100% - Test 4.2.....	165
Figure 147 – The beam's mid-span deflection vs. time EX 4-19.1 100% - Test 4.2.....	165
Figure 148 – Load Ratio vs Time - EX-4-12.7-60%, Test 3.1	167
Figure 149 - Load Ratio vs Time - EX-4-12.7-100%, Test 3. 2	167
Figure 150 – The steel column's temperature vs. time EX 4-12.7-60%.....	169
Figure 151 – The steel column's temperature vs. time EX 4-19.1-100%.....	169

Figure 152 – The bolts and plate temperature vs. time EX 4-12.7-60% - Test 3.1	170
Figure 153 – The bolts and plate temperature vs. time EX 4-12.7-100% - Test 3.2	171
Figure 154 – The bolts and plate temperature vs. time EX 4-19.1-60%- Test 4.1	172
Figure 155 – The bolts and plate temperature vs. time EX 4-19.1-100% - Test 4.2	172
Figure 156 - Bolts bending a) test 3.1, b) test 3.2	173
Figure 157 – The steel plate and bolts EX 4-19.1 60% Test 4.1	173
Figure 158 – The wood's temperature vs. time at the connections Test 3.1	175
Figure 159 – The temperature distribution along beam width vs. time- Test 3.1	176
Figure 160 – The wood's temperature vs. time at the mid-section - EX 4-12.7-60% - Test 3.1	177
Figure 161 – The temperature distribution along the beam's depth vs. time EX 4-12.7-60% - Test 3.1	178
Figure 162 – The temperature distribution along beam's width vs. time - Test 3.1	178
Figure 163 – The wood's temperature vs. time at the connections EX 4-19.1-100% - Test 4.2	179
Figure 164 – The wood's temperature vs. time at the mid-section - EX 4-19.1-100%- Test 4.2	180
Figure 165 – The charring rates at the mid-span and the connection for Groups 3 & 4.	181
Figure 166 - failure mode, Test 3.1 - a) wood splitting, b) bolt holes elongation, c) bolts bending	182
Figure 167- The wood splitting propagated to mid-span - Test 3.2	183
Figure 168 - a) wood splitting , b) bolts hole elongation and bolts bending- Test 3.2 ...	183
Figure 169 – The wood splitting at one side - Test 4.2	184
Figure 170 - No bending of the bolts - Test 4.2	184
Figure 171 - Bolt temperature vs. time – a) T10 and b) T11 Group 3	188
Figure 172 -The wood's section temp vs. time - a) T24, b) T25, Group 3	189
Figure 173 - Test vs. model temperatures at Points T18 and T20 for Group 4 from Ohene (2014) [86]	190
Figure 174 - EX Test vs the model's temperature at Point T16 for Group 4 Ohene (2014) [86]	190

Figure 175 – The temperature distribution a) model at mid-span and connection b) tests at mid-span c) test near connections	192
Figure 176 – The crack generated at bolt's edge	193
Figure 177 – The progressive damage of the Exposed connections in Group 3.....	193
Figure 178 - Strains on the bolts exceed the steel's yielding strain at 51.1 min.....	194
Figure 179 – The progressive damage of the EX connection for Group 4, from Ohene [86].....	194
Figure 180 – The failure times of the Exposed connections for Group 3 and 4; Test vs. Model	195
Figure 181 – The time to failure vs. the load ratio of the Concealed connection for Groups 3 and 4	197
Figure 182 – The mid-span deflection vs. time for Group 3	198
Figure 183 – The mid-span deflection vs. time for Group 4	199
Figure 184 -The effect of bolt diameter on the failure time for Groups 3 and 4	201
Figure 185 – The mid-span deflection vs. time for Groups 3 and 4	201
Figure 186 – The beam's mid-span deflection vs. time SE 1-12.7 100% - Test 5.2	203
Figure 187 – The beam's mid-span deflection vs. time SE 1-12.7 60% - Test 5.1	203
Figure 188 – The beam's mid-span deflection vs. time SE No Bolts- 60% - Test 5.3 ...	204
Figure 189 – The beam failure Test 5.1 a, b. and c.....	206
Figure 190 –The beam failure Test 5.2 a, b, and c.....	207
Figure 191 – The steel plate failure Test 5.2 a, and b.....	208
Figure 192- Seated Connection Assembly without the bolts.....	210
Figure 193 – The wood's section temp vs. time - a) T25, b) T19, Group 6	212
Figure 194 – The damage and failure at 1/3 of the beam span	213
Figure 195 - Yielding of the steel plate	213
Figure 196 – The progressive damage of the SE Connections a and b.....	214
Figure 197 – The failure time of the Seated connections for Groups 5 and 6; Test vs. Model	215
Figure 198 – The failure time vs. the load ratio of the Seated connections for Groups 5 and 6.....	217
Figure 199 – The mid-span deflection vs. time for Group 5	218

Figure 200 - The effect removing the bolts on the failure times for Groups 5 and 6	219
Figure 201 – The Exposed connections with the two bolts - Group 6.....	220
Figure 202 – Mid-span deflection of the beam vs. time CN 2-12.7 100% - Test 6.1	221
Figure 203 – Mid-span deflection of the beam vs. time EX 2-12.7 100% - Test 6.2	221
Figure 204 – The typical failure mode for the Exposed connections with 2 bolts	223
Figure 205 - 2 bolt connection model assembly	224
Figure 206 –Wood splitting as predicted by the model for the 2 bolt connections	225
Figure 207 –The typical progressive damage curves of the CN Connection with 2 bolts	225
Figure 208 – The bolts yielding as predicted by the model for the 2 bolt connections ..	226
Figure 209 – Failure time vs. the load ratio of the CN connections using two bolts.....	228
Figure 210 – The failure time vs. the load ratio of the EX connections with two bolts ..	228
Figure 211 - The effect the bolt diameter had on the failure time for the CN connections using 2 bolts	230
Figure 212 - The effect of bolt diameter had on the failure time for the EX connections using 2 bolts	230
Figure 213 – The effect of the number of bolts on the failure time of the CN connections	232
Figure 214 – The effect of the number of bolts on the failure time of the EX connections	234
Figure 215 – Failure times for the SE, EX, and CN connections using 4- 19.1- mm bolts	236
Figure 216 – Failure times for the SE, EX and CN connections with 4-12.7-mm bolts	238
Figure 217 – Failure times for the different types of connections.....	240
Figure 218 - Curve fitting analysis results for the CN connections.....	244
Figure 219 – The tested or modeled time vs. the formula predicted time for the CN connections	245
Figure 220 - Curve fitting analysis results for the EX connections	246
Figure 221- The tested or modeled time vs. the formula predicted time for the EX connections	247
Figure 222 - Curve fitting analysis results for the SE connections with the bolts.....	248

Figure 223 -Curve fitting analysis results for the SE connections without the bolts.....	249
Figure 224 – The tested or modeled time vs. formula predicted time for the SE connections with bolts.....	250
Figure 225 - Tested or modeled time vs. formula predicted time for the SE connections without bolts.....	250
Figure 226 -Curve fitting analysis results for the SE connections with and without bolts	251
Figure 227 - Tested or modeled time vs. formula predicted time for SE connections with and without bolts.....	252
Figure A Concealed plate details	262

1 Introduction

1.1 Problem Statement

The use of wood, as a building material, dates back to the early days of mankind. It has been used successfully for thousands of years in construction. In Asia and many other parts of the world there are many wooden buildings that have withstood the passage of time, a testament that wood is durable and long lasting. The use of wood however, because of its combustible nature, has been restricted as a construction material by the National Building Code of Canada (NBCC) [1] and other building codes around the world. But with the introduction of performance/objective-based codes along with the recent interest by building designers and architects to implement sustainable building materials, the use of wood in construction is on the rise.

Two common construction categories are used in the assembly of timber structures: light framing and heavy timber. Small dimension timber is used in light framing construction and large dimension timber is used in heavy timber construction. The minimum dimensions of wooden structural elements in heavy timber construction are specified by the NBCC [1] in Table 3.1.4.6 of the code.

The light framing system is typically used in residential buildings and is normally protected by gypsum boards to enhance its fire performance. The capability of gypsum boards to absorb large amounts of heat (to evaporate the water molecules) gives exceptional fire resistance for light framing structures by increasing the time required to raise the temperature of the timber behind the gypsum board.

Heavy timber is generally used in non-residential buildings and the large section of the timber allows for adequate fire resistance due to the slow charring rate (about 0.6 to 0.8 mm / min under direct fire exposure) [2] to [10]. The produced char layer isolates the timber from the ambient temperature and reduces the egress of oxygen and the direct contact between the flame and the inner layers, which further reduces the charring rate. Both light framing and heavy timber systems fall under the combustible construction category as per the NBCC [1].

The NBCC places a restriction on the number of stories allowed to be constructed with combustible building materials depending on the type of occupancy, building area, number of facing streets, fire resistance and the availability of sprinkler systems.

However for non-combustible construction materials the NBCC does not apply restrictions of building height if all the code requirements are met. The use of wood in larger and taller buildings necessitates its combination with other building materials, such as concrete and steel for optimum performance. This type of construction is called the 'Hybrid Building System' and has become increasingly popular in the construction of mid-rise buildings (structures between 5 and 12 stories). The hybrid building system utilizes a mixture of materials such as heavy timber and concrete or steel.

Typically any building construction system consists of several members connected to one another, e.g. columns, beams, walls and floors. Light framing systems use fasteners such as nails, screws and light-gauge steel plates. While dowels, bolts and steel plates are used in heavy timber connections.

These are the weakest links in heavy timber construction, particularly under fire conditions. Building codes require that the connections' capacity is equal to or greater than the factored load applied on the connections. This also safeguards the overall stability of the structure. Building codes require that connections must have the same fire resistance as the parts they connect.

However, if exposed to fire, the temperature of the metal fasteners rises faster than the timber due to the high thermal conductivity of steel and this may lead to a reduction in the capacity of the fastener. Moreover, the fastener also increases heat transfer from the exposed surfaces to the inner layers of wood, causing charring, which further reduces the capacity of the connection.

The connections used in a building system consist of different materials. Steel and concrete columns are connected to timber beams using metal fasteners in varied geometries and arrangements. The specific physical properties of steel, concrete and wood cause them to behave differently when exposed to fire for a prolonged duration. The char layer, produced by timber at an elevated temperature, has distinct characteristics from its parent wood. These factors serve to complicate the overall analysis of the fire performance of the connections used in hybrid timber construction. Unfortunately, this issue has received very little attention and has rarely been tested in full-scale tests.

The NBCC allows the use of heavy timber for combustible construction if a fire resistance of 45 minutes is required. The code gives general guidance for the design of heavy timber connections such as; the minimum thickness of the timber, end and loading distances, the proper amount of space needed between fasteners and fastener's rows and

columns. However, detailed information or methods used to calculate the fire resistance for hybrid timber connections cannot be found in the NBCC. Basic parameters are provided for the calculation of glulam beam and column sections for fire resistance. Yet, no guidance related to the design or selection of hybrid connections is given.

1.2 Previous Work in this Field

Design of timber connections in ambient conditions is covered by codes for both brittle and ductile failure mode (Quenneville and & Mohammad) [11] (Quenneville and & Mohammad) [12]. Researchers have studied the brittle failure modes of timber connections in an effort to improve the code equation for predicting connection capacity. Guan [13], Moses [14] and Hong [15] studied the failure modes of connections using finite element methods.

Designers typically design timber connections in compliance with building code requirements to ensure the capacity of the supporting frame including the connections is similar or exceeds the design capacity of the connected members or the factored applied loads on the members, to ensure stability of the structure. If fire resistance is required; codes require that the supporting frame including the connections shall have the same fire resistance as the supported assembly.

In recent years, a number of research efforts have been devoted to investigating the fire performance of timber connections and developing calculation methods for estimating the fire resistance of timber connections using different connection configurations and various types of dowels Noren [16], Diham [17], Kruppa [18], Ayme [19], Laplace [20], Lau [21], Chuo [22], Frangi [23] and Koing [24]. Carsten [25] studied the influence of

steel elements on the temperature distribution in timber connections using multiple shear steel plates.

More recently, experimental and numerical work on timber connections has been done at Carleton University resulting in correlations for determining the fire resistance of steel-wood-steel (SWS) and wood-steel-wood (WSW) connections subjected to tensile loading (Peng) [26]. The performance of hybrid timber connections in fire conditions, has received little attention from researchers. The analysis of hybrid timber connections is complicated due to the different properties of steel, concrete, and wood and their unique behaviors when exposed to intense heat. As a result, very little knowledge exists on the behavior and capacity of these connections when subjected to fire.

Researchers have partially covered the behavior of hybrid connections at ambient conditions using theoretical, numerical and experimental analysis. Sebastian [27], studied the path dependency of ductile and brittle connections in hybrid structures using Finite Element analysis, simulating the composite section of a steel beam connected to a concrete slab via steel studs. Joao [28], used numerical methods to investigate the behavior of hybrid connections between a GFRP (Glass Fiber Reinforced Polymer) beam and a concrete slab connected using stainless steel bolts or a continuous epoxy adhesive layer applied on the interface between the two materials. He also carried out experimental tests and used the results to validate the finite element model.

Sudhakar [29], also used a finite element model validated by experimental tests (carried out by others) to investigate the behavior of hybrid connections under cyclic loading. The tested frame consisted of precast concrete beams and columns connected by a steel angle

and a steel plate. Seug [30], used the same approach to validate the FE model (ABAQUS) when studying the behavior of hybrid connections between steel and concrete beams using headed stud connectors. Hawileh [31], used limited experimental tests to validate the FE model ANSYS to predict the response and behavior of precast hybrid concrete beam – column connections under cyclic loads. Gustavo [32], carried out four full scale tests to investigate the behavior of hybrid connections between concrete columns and beams (steel truss encased with FRC (Fiber Reinforced Concrete)) under seismic loads. Faggiano [33], in his effort to develop an innovative steel connection for the retrofit of timber floors in ancient buildings, numerically investigated the behavior of hybrid connections between a timber beam and a concrete slab using steel sections. Jorge [34], carried out experimental analysis of laterally-loaded nailed timber – to concrete connections to assess the methodology proposed in the Eurocode 5 [35], for laterally-loaded nailed timber-to-concrete connections.

No investigations were found in the literature review, concerning the behavior of hybrid connections when exposed to fire, in particular, between materials such as wood and steel or wood and concrete.

The change in thermal and mechanical properties of wood and steel significantly affects their structural integrity and load bearing capacity. Although this topic has received a lot of attention, researchers have arrived at different conclusions concerning the specific values for the mechanical and thermal properties of wood, (Buchanan) [3], and steel, (Eurocode 5) [35], at elevated temperatures.

1.3 Scope and Objectives

This study investigates the fire performance of hybrid connections between steel columns and a glulam beam. The three types of connections considered are, the Concealed shear tabs, the Exposed shear tabs, and the Seated beam connections.

The objectives of this study are:

- To better understand how certain parameters affect the structural and fire performance of wood-to-steel hybrid connections.
- To develop a 3-D heat transfer model coupled to a finite element program for structural analysis using ABAQUS.
- To use the validated computer models of the different hybrid timber connection types to predict the impact of various parameters.
- To produce new experimental data of heat transfer between the different components of hybrid timber connections when exposed to fire.
- To understand the different types of failure for hybrid timber connections when exposed to fire.
- To develop simple methods for the design of hybrid timber connections based on the experimental findings and numerical predictions for eventual submission to code committees for potential incorporation into future building codes.
- To provide suggestions concerning the acceptability of certain types of hybrid timber connections for fire resistance in the NBCC [1] and the Canadian wood design standard (CSA O86) [36].

1.4 Methodology

Experimental tests and computer modeling were conducted to investigate the fire performance of hybrid timber connections between a timber beam and steel columns. Variables, which affect the performance of the connections, such as type (the Concealed, the Exposed and the Seated), number and size of bolts, and load ratio were examined during the experimental tests. All specimens were exposed to the effects of the CAN/ULC S101-07 [37] standard time-temperature curve.

The finite element program ABAQUS [38] [39] was used to develop thermal and structural models of hybrid timber connections to simulate the experiments. One of the features of this program is the capability to simulate transient heat conduction in and between building materials such as wood and steel as well as the complexity of heat exchange between them. ABAQUS also accounts for changes in material mechanical and thermal properties under elevated temperature. The structural model was coupled with the heat transfer model to simulate the temperature distribution in the hybrid connections considering conduction in solids and convection and radiation on the exposed surfaces under standard fire conditions.

The thermal and mechanical properties of materials required to develop the structural and thermal models for this study were based on data provided by previous researchers in this field [2] [3] [6] [40] [41] [42] [43] [44] [45] [46].

The furnace temperature used was the standard time-temperature fire exposure (CAN/ULC-S101-07) [37], which was also used as the boundary condition for the thermal models. The computer models were validated using the full-scale test results.

2 Literature Review

The literature reviewed focused on fire resistance, the mechanical and thermal properties of steel and wood at normal and elevated temperatures, and previous research on hybrid connections at ambient temperature conditions.

2.1 Fire Safety

Fires cause massive damages to property and threaten the safety of occupants in buildings and residences. The primary goal of fire protection is to minimize the probability of death, injury, property loss and environmental impact that result from fires. (Buchanan) [3]. Building codes emphasize the necessity of providing multiple safe exits as well as a reasonable amount of time for occupants to escape. These codes also advise how best to prevent the spread of fire to adjacent compartments and/or buildings. From a structural perspective, building codes require that structural elements have sufficient fire resistance to maintain stability and prevent collapse. In other words, the fire resistance of a building should be greater than the severity of the fire (NBCC) [1].

Fire severity is a measure of the destructive impact of a fire. It can be quantified in terms of time, temperature, or strength, although it is usually measured in units of time (Buchanan) [3].

Fire resistance is the capacity of a structural element to withstand the fire severity without failure or collapse. The fire resistance is determined based on the time a structural element can withstand a standard fire exposure and satisfy specific criteria imposed by standard fire tests. The assigned time measured from the standard test rounded to the

nearest hour or fraction thereof is known as the element's fire resistance rating (CAN/ULC-S101-07) [37].

The Canadian standard CAN/ULC-S101-07 [37], provides full details related to the procedure, criteria and the time-temperature curve to be used in the standard fire test.

The Canadian standard time-temperature curve is similar to the ASTM E119 [47] and the ISO 834 standard [48], as shown in Figure 1.

Figure 1 - Standard time temperature curve

Although fire tests are the most direct method for determining the fire resistance of a system, they are expensive and time consuming. An alternative approach is using analytical or numerical methods for heat transfer and structural modeling. These methods however, should be verified and validated by using the results of full-scale tests before they can be used to replace tests.

2.2 Behavior of Wood and Steel in Timber Connections

The behavior of timber connections, consisting of wood connected by steel fasteners, at ambient and elevated temperature attracted the attention of several researchers. However, fewer studies have been conducted concerning connections between steel and wood at ambient conditions. Moreover, virtually no research has been done on the behavior of wood and steel connections at elevated temperatures.

2.2.1 Behavior at Ambient Temperature

Steel is an isotropic material with a well-defined stress strain relationship. The steel plates and fasteners used in a connection are assumed to behave as an elasto-plastic material. At ambient conditions, steel does not experience a great degree of deformation that leads to what is known as ‘steel yielding’ (Kulak and Grondin) [49].

Wood is an orthotropic material with different properties related to its three principle axes with respect to the grain direction (longitudinal, radial and tangential) as shown in the Figure 2 (CSA O86) [36].

Figure 2 Three principal axes in wood (CSA O86) [36]

The major strength of wood is in the longitudinal direction (parallel to grain), which is greater than the radial (perpendicular to grain) as illustrated in the stress strain relationship in Figure 3 (Buchanan) [3]. A minimal difference is observed in the strength between the radial and tangential (tangent to the grain) directions (Peng) [26].

Figure 3- Stress Strain relationship for wood (Buchanan) [3]

Figure 3 (above) shows the wood stress-strain relationship, which has ductile behavior in compression and brittle behavior in tension due to wood splitting when the stress reaches and exceeds the wood tensile capacity.

There are two classifications of failure in wood connections, catastrophic or brittle failure and non-catastrophic or ductile failure. The use of bolts in wood connections increases the possibility of brittle (splitting) failure; a brittle failure involves fracture of the wood around the connection. In a perfectly brittle failure, tension in the wood causes it to crack

before any ductile behavior occurs. Once a crack develops, the strength of the connection is rapidly lost causing immediate failure. Ductile (bearing) failure happens when the wood is crushed under the bolt shank as stress reaches the plastic state.

Pure brittle and pure ductile behaviors are rare, with the majority of bolted connections failing due to a combination of both. In these connections, some ductile behavior occurs first, and then, with increasing load, the wood member fails due to brittle fracture (Habkirk & Quenneville) [50].

Bolts transfer shear forces to the wood member by bearing on wood fiber. If the load is applied parallel to grain, as in a tension connection, a compressed portion underneath the bolts produces lateral stresses perpendicular to the grain that leads to splitting. Yet, if the load is applied perpendicular to the grain as in the case of beam to column connections / shear connection, splitting failure is more likely to occur (Rodd) [4], (Jorissen) [51] and (Sjodin) [52]. The possible failure modes are summarized in Figure 4 ,(CAN/CSA O86-09) [36].

Yielding

(a) All loading directions

Row shear

Group tear-out

Net tension

(b) Parallel-to-grain Q_f

Splitting

(c) Perpendicular-to-grain**Figure 4 Typical failure modes of wood connections (CAN/CSA O86-09) [36]**

2.2.2 Potential Failure Modes

The Canadian design code (CAN/CSA O86-09) [36], for bolted timber connections takes into account both ductile and brittle failure modes. The ductile mode of failure included in the European code [35] was introduced by Johansen [53] and modified by Larsen [54].

The Canadian code (CAN/CSA O86-09) [36] requires that for joints loaded perpendicular to the grain, the joints should be designed to satisfy the following requirements:

$$Q_f \leq Q_r \quad \text{Equation 1}$$

$$N_f \leq N_r \quad \text{Equation 2}$$

where:

Q_f : is the factored load perpendicular to the grain

Q_r : is the factored splitting resistance

N_f : is the factored load on the joint

N_r : is the factored lateral yielding resistance

The splitting resistance is calculated as follows:

$$Q_r = Q S_{rT} \quad \text{Equation 3}$$

$$Q S_{rT} = \sum Q S_{ri}$$

$$Q S_{ri} = \phi_w Q S_i (K_D K_{SF} K_T)$$

$$Q S_i = 14 t \sqrt{\frac{d_e}{1 - \frac{d_e}{d}}}$$

t : is the side member's thickness, mm

d_e : is the wood member's effective depth, mm

d : is the total depth of the wood member, mm

ϕ_w : is the resistance factor for brittle failures = 0.7

K_D : is the duration of the load factor

K_{SF} : is the service condition factor

K_T : is the fire retardant treatment factor

For ductile failure, the Canadian code adopted the European code ductile mode of failure and defined the possible ductile failure modes at ambient temperatures for double-shear connections loaded parallel to grain as illustrated in Figure 5 (CAN/CSA O86-09) [36]:

Figure 5- Ductile failure modes for double shear timber connections (CAN/CSA O86-09) [36]

The factored lateral yielding resistance is:

$$N_r = \phi_y n_u n_s n_f \quad \text{Equation 4}$$

ϕ_y : is resistance factor for yielding failures = 0.8

n_u : is unit lateral yielding resistance, N

n_s : is number of shear planes in the joint

n_f : is number of fasteners in the joint

The unit lateral yielding resistance n_u shall be taken as the smallest value from the following expressions:

- For failure mode I: $n_u = f_1 d_f t_1$

- For failure mode II: $n_u = \frac{1}{2} f_2 d_f t_2$

- For failure mode III: $n_u = f_1 d_f^2 \left[\sqrt{\frac{1}{6} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_1}{d_f} \right]$

- For failure mode IV: $n_u = f_1 d_f^2 \left[\sqrt{\frac{2}{3} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} \right]$

where:

f_1 : is the embedment strength of side member, Mpa

f_2 : is the embedment strength of main plate, Mpa

f_y : is the bolt's yield strength, MPa

t_1 : is the side member's thickness, mm

t_2 : is the main member's thickness, mm

d_f : is the fastener's diameter, mm

For wood member embedment strength:

$f_1 = 22 G (1 - 0.1 d_f)$ For fastener bearing perpendicular to grain

G : is mean relative density

For steel member embedment strength:

$$f_2 = 3 f_u \left(\frac{\phi_{steel}}{\phi_y} \right)$$

f_u : is the ultimate strength of steel grade, MPa

ϕ_{steel} : is the resistance factor for metal member = 0.67

ϕ_y = is the resistance factor for yielding failures = 0.8

2.2.3 Hybrid Connections at Ambient Conditions

The performance of hybrid connections is complex due to the different materials involved and this complexity increases under fire exposure. Several researchers have studied hybrid connections using experimental tests and numerical modeling (Joao) [28], (Parra-Montesinos, Dasgupta, & Goel) [32], (Hawileh, Rahman, & Tabatabai) [31], (Kim & Nguyen) [30], experiments only (Branco, Cruz, & Piazza) [34], and numerical modeling only (Sebastian) [27], (Sudhakar A Kulkarni, Li, & Yip) [29].

The flexural behavior of hybrid connections between glass fiber reinforced plastic (GFRP) beams and concrete slabs, was investigated by Joao [28], in an effort to study the effect of two different shear connection systems, (i) stainless steel bolts, fixed on both sides of the GFRP profile's top flange and embedded in the concrete layer (bolted solution) and (ii) a continuous epoxy adhesive layer applied on the interface between the two materials (bonded solution) (Figure 6).

Figure 6- The GFRP - Concrete composite section: bolted (left) and bonded (right)- Joao [28]

The test samples were supported in a simple manner and were subjected to a point load at mid-span as well as two point loads applied at about 1/3 span.

Joao [28], used the ANSYS software to develop a 3-D FE model using the 3-D 8-node 64SOLID element, which has three degrees of freedom per node (translations in the nodal directions). Symmetry boundary conditions were utilized. All of the tested materials were modeled assuming linear-elastic behavior. The GFRP profile was modeled as an orthotropic material; while the adhesive and the concrete were modeled as isotropic materials. Joao found out that a continuous epoxy layer provided a higher degree of rigidity than that obtained using steel bolts.

Gustavo [32], conducted research concerning another type of hybrid connection between fiber reinforced concrete (FRC) encased steel truss beams and reinforced concrete (RC) columns. His objective was to develop a new connection design that would allow the use of FRC-encased steel truss members in earthquake-resistant RC framed construction.

The experiment consisted of testing four beam-to-column connections to evaluate the seismic behavior of the various connection schemes between the various components.

The connections were designed to remain elastic during a seismic event, forcing inelastic deformations to occur in the hybrid beam region near the transverse steel tube.

To achieve moment transfer in the beam-column connection, Gustavo [32], used external steel rods connected to the embedded truss chord. Shear force transferred through the use of a bolted connection between the steel truss and the RC columns. The connection scheme used is shown in Figure 7, Figure 8 and Figure 9.

Figure 7 - Gustavo test frames [32]

Figure 8 - Gustavo test frames [32]

Figure 9 - Gustavo's proposed connection concept [32]

The connection configuration used by Gustavo performed well under large displacement reversals. The connection elements behaved elastically as intended forcing inelastic rotations to occur in the beam region near the anchorage location of the external rods and exceptional energy dissipation capacity was observed in the test specimens.

Sudhakar [29], developed a finite element model to study the effect of a number of variables on the behavior of hybrid steel-concrete connections under cyclic loading. He was able to validate the model using full-scale test results carried out at Nanyang Technological University, Singapore. The experimental tests investigated the effect of axial load on columns, the connection plate thickness, and the continuation of beam bottom reinforcement. The details of such typical connections are shown in Figure 10.

Fig. 1. Beam-column joint with cross section details.

Figure 10 - Sudhakar typical connection details [29]

The test results showed that the axial loading was a critical parameter in the studies of beam–column joints, and was beneficial to the joint shear resistance. However, experimental data showed that some specimens failed due to the pullout of the embedded beam bottom bars instead of joint shear failure. The tests also showed that the connection plates adjacent to the joint played a key role in transferring the moment and shear between the column and beams.

It was observed that increasing plate thickness not only increased the energy dissipation, but also helped in the smooth distribution and reduction of the maximum principal stresses with the ductility level of the joint remaining constant. An improved energy dissipation and higher ultimate strength was observed in the specimens with increased plate thickness at the connection.

Comparison of the analytical and experimental results of all the specimens showed that the results obtained from the FE analyses were quite similar to the experimental observations. Furthermore, the failure modes and the ultimate ductility capacities correlated well with the experimental results.

The FE analyses also showed that the results of the deformations and cracking patterns matched well with the experimental observations. Therefore, the use of FE modeling techniques can be used to further study the joint performance by varying other parameters. Figure 11 shows the FE model at the joint.

Figure 11 - Sudhakar's FE model at joint region [29]

The behavior of hybrid connections under cyclic loading was also studied by R.A Hawileh [31]. He used the results of experimental tests to develop and validate a nonlinear finite element model using ANSYS 8.0 to predict the response and behavior of precast hybrid beam–column connections subjected to cyclic loads.

The finite element model was developed to accurately represent the geometrical configuration and dimensions of the test specimen. The hybrid connection was modeled with dimensions and reinforcement details as illustrated in Figure 12 and Figure 13.

In the experimental setup, the parameters were as follows: the frame was pinned at the bottom of the column and roller supported at the top of the column and the end of the beam as shown in Figure 14. These boundary conditions represented the experimental setup shown. Nodes at the bottom of the column were restrained in all translational degrees of freedom, and the beam's edge was restrained in the vertical y and lateral z

Figure 14 - R.A Hawileh test frame and FE model [31]

The FE model proved effective in producing results that were in agreement with the experimental data with respect to the expected elastic and plastic ranges. The response envelope from the finite element analysis correlated fairly well with the experimental

results. The development of the FE model allowed for additional investigation of such connections under other load bearing conditions and different design parameters.

Seung [30], used a nonlinear three-dimensional finite element model using ABAQUS to investigate the behavior of hybrid steel-pre-stressed concrete (PSC) beam connection.

Samples were used from previous research to compare with the results of the proposed FE modeling. Six components, including concrete beams, rebar, tendons, a steel beam, headed stud connectors, and rigid support were modeled separately as shown in Figure 15, and assembled to make a complete specimen model. Both geometric and material nonlinearity were included in the finite element analysis.

Figure 15 -Seung FE mesh of the beam specimens [30]

The FE model was verified against the experimental data. The specimen used in the experiment was a hybrid-steel PSC beam composed of three beam parts as shown in Figure 16. The full beam length was 3.9 m resting on supports of 3.5 m span. The two PSC beams at both sides were connected to the steel segment at the middle.

Figure 16 – Seung’s details of the beam specimen [30]

The study showed that the proposed FE modeling can accurately capture all the main features of the behavior of the steel-PSC beam connection.

In an effort to assess the accuracy of the methodology proposed in Eurocode 5 for laterally loaded nailed timber-to-concrete connections, Jorge [34], analyzed and discussed the adequacy of the methodology proposed by Eurocode 5 for the dowel-type fasteners. Figure 17 shows the experimental specimen.

Figure 17 – Jorge's composite timber - concrete joint configuration [34]

In the first part, the calculation methodology established by Eurocode 5, Part 1 for the timber-to-timber nailed connections was analyzed. Afterwards, a similar comparison process was carried out for timber-to-concrete connections using Eurocode 5, Part 2 methodologies, which showed the mechanical behavior of the composite structures. For instance, composite timber-concrete elements are more often influenced by the joint slip modulus than the joint's ultimate load-carrying capacity. Jorge recommended the degree of strength and stiffness of composite timber-concrete connections must be evaluated by experimental tests. He concluded that the European yield model, used in Eurocode 5, was able to predict with reliability the failure mode of the connection. Nevertheless, the results of tests showed significantly different values from those obtained using the procedure defined in Eurocode 5.

The experimental results obtained show that a better approximation of the mechanical behavior can be reached if a different formulation for the slip modulus of timber-to-concrete connections is considered.

2.2.4 Hybrid Connections in Fire

The performance of hybrid timber connections under fire conditions has received little attention from researchers in the field, especially regarding wood-to-steel hybrid connections.

Ing [53], investigated how the width of a wooden beam affects the temperature at the location of mechanical connectors in wood-to-concrete hybrid connections. The connectors were placed in the middle of the timber beam. The test specimens were composed of glued laminated timber beams of widths equal to 80 mm, 120 mm and 160 mm. The beam height was 160 mm, and an 80-mm thick concrete slab was constructed on 20-mm thick decking (see Figure 18).

Figure 18 – Ing’s Timber - concrete composite beam [53]

Ing [53] developed a FE numerical model to study the effect of beam width on the rate of heat transfer into a wooden beam which was exposed to fire. During this experiment, the temperature applied was comparative to gas temperatures measured in a fire compartment during a fire experiment in a real building. Ing found out that the temperature non-

uniformity along the connector axis increased as the initial beam width decreased. He also found that the influence of natural fire on temperatures inside the timber beam was very similar to that during a standard fire. However the temperature rose more quickly in the scenario of a natural fire.

Schnabl [56], modeled the behavior of timber composite beams with interlayer slip, when they were simultaneously exposed to static loading and fire. Figure 19 shows the beam composite. Schnabl used the two-dimensional equations developed by Luikov [57], to simulate heat, moisture and mass transfer in timber. The mechanical behavior of a timber composite beam was modeled using Reissner's [58], kinematic equations.

Figure 19 – S. Schnabl descriptive geometric of two layer timber composite beam [56]

Schnabl evaluated the charring depths of solid timber beams and compared the results to the published charring rates in the literature. He compared the results of the proposed charring model with the empirical charring models from the literature and concluded that the results of the proposed model were slightly different from the results published by Lawson's and Schaffer's [59] models at the beginning, but were virtually equal at 60 min. Similarly, the model proposed by Eurocode 5, White's and Nordheim's [60] model, and the model proposed in that paper, differ considerably during the initial 15 minutes, but

later the results were similar to Schaffer's experiment. The temperature gradients were the highest at the boundary between the timber and the char. Charring occurs most quickly in the first 5 minutes and due to the insulating properties of char, charring gradually decreases with time. Since the corners of beams were subjected to heat transfer from two directions, charring originated there. As a result of charring, the load-bearing cross-section begins to round and loses its original rectangular shape.

Erchinger [23], investigated how the presence of steel elements affects temperature distribution in timber. Specifically, he studied the effect of fire on the structural integrity of multiple shear steel-to-timber connections with slotted-in steel plates and steel dowels. Connections under tension were tested parallel to the grain with different plate thicknesses, dowel lengths, and fastener diameters. Each connection variation was exposed to the ISO-fire. The finite element package ANSYS Workbench (in combination with ANSYS Release 10.0) was used to simulate the connections. Erchinger showed that due to the high thermal conductivity of steel, the heat flux through the steel plates and the dowels led to higher temperatures inside the timber. The residual cross-section of the timber member was measured by laser scanning. This method was a quicker and more precise alternative to the manual method. It was also observed that moisture content had no significant influence on the temperature development. In addition, Erchinger found that a difference of 1 mm gap, filled with air, between the timber slot (6 mm) and the plate (5mm) had no influence on the results.

Peng [26], studied the performance of steel-wood-steel (SWS) and wood-steel-wood (WSW) connections under tension when exposed to a standard fire. The fasteners used were 12.7 mm and 19.1 mm bolts. A three-dimensional numerical heat transfer model

and an analytical structural model were used to simulate the temperature profiles in the timber connections and the strengths of the timber connections at elevated temperatures. The test results showed that all the tested WSW connections using bolts as fasteners with no protection had a fire resistance of less than 45 min. Under the same test conditions, the SWS connections were found to have a fire resistance of less than 25 min. When protective membranes were used on the WSW and SWS connections, the fire resistance increased by about 15 to 30 minutes.

2.2.5 Numerical Modelling

Full-scale tests are ideal to study the behavior and failure of structures used in buildings. However, they can be costly and time consuming. They require adequate facilities, space, setup, and manpower. If planned and executed properly, computational models are an attractive alternative to expensive investigations.

In recent years, nonlinear finite element methods (FEM) have been used to study the mechanical performance of wood joints affected by specific variables [13] [14] [26] [61] [62]. To develop a finite element model for timber connections, constitutive models of materials, contact algorithms and failure criteria are required.

In general, steel can be modeled as an isotropic elasto-plastic material while wood is usually assumed to be a homogenous and orthotropic continuum. During numerical analysis, this assumption is often expanded to transversely isotropic materials. They possess identical properties in both the tangential and radial directions. The manner in which wood commonly responds to compression stress at the contact area under a

fastener is a nonlinear compressive response; therefore, researchers tend to model wood as an elasto-plastic material in compression. (Peng) [26].

Researchers used 2-D and 3-D Finite Element models to study wood behavior using different methods to model the nonlinearity of wood such as considering biaxial compression (Karouf) [62], changing the tangent modulus with relation to the stress strain regions (Rahman, Chiang, & Rowlands) [63], or assigning different tangent moduli. A tri-linear stress-strain relation was used to develop nonlinear behavior of wood in compression and shear by Patton-Mallory [61]. Peng [26], used an analytical structural model, combined with temperature profiles calculated using the heat transfer model, to predict the fire resistance of timber connections.

Two types of failure which can occur in wood as a result of high tension and shear are; ductile compression failure and catastrophic brittle failure. In order to test the brittleness in timber connections, researchers adopted two major criteria.

The first, limit criteria, considers individual stress or strain, without considering the combination effects of the stresses or strains, the maximum stress or strain is the most common criteria used by Patton-Mallory [61], to model 3-D single bolt connections.

The second is the interactive criterion, where failure depends on more than one component of stress or strain. This was used by other researchers [46] [64].

2.3 Wood Properties at Elevated Temperature

The degree to which timber members are able to resist fire depends on factors such as species of wood, density, moisture content, and wood section. At elevated temperatures, wood suffers thermal decomposition which is a continuous process until

wood becomes char at a temperature of about 280°C to 300°C (Buchanan) [3]. The char layer has no strength, but it isolates the core of the wood and reduces heat transfer from the fire to the wood core. The speed at which the char layer moves into the wood is known as the ‘charring rate.’ Many researchers reported charring rates of wood around 0.6 – 0.7 mm/min [5] [6] [25] [59] [65]. Similar rates had been specified by standard codes such as CAN/CSA-O86-1 [36] and Eurocode-5 [66].

The CSA code [36] specified the minimum dimensions for heavy wood members to be used to meet the requirement of 45 min. fire resistance. Timber construction is considered heavy when the dimension of beams and columns, and decks exceed 150 mm and 50 mm respectively (Buchanan) [3].

2.3.1 Wood Thermal Conductivity

Wood has different thermal conductivities along its major axes; radial, tangential and longitudinal. Researchers have explored the effect of elevated temperature on the thermal conductivity of different wood species [2] [5] [24] [26] [67] [68] [69] [70] [71]. Peng [72] summarized those results in their entirety, with respect to thermal conductivity values (perpendicular to grain) . Figure 20 shows that the conductivity increases up to 200°C and then decreases between 200°C to 350°C, due to the production of char. After 350°C, the thermal conductivities start to rise.

Figure 20 - Thermal conductivity of wood at elevated temperatures [26]

Researchers also found that the tangential conductivity is a little bit higher than the radial conductivity, the ratio ranging from 0.9 to 0.97, while the longitudinal conductivity is higher than the transverse by a ratio of about 1.5 to 2.8 [67].

2.3.2 Specific Heat

Peng [72], summarized the specific heat values reported by several researchers [73] [74] [69] [75] [41] [71] as shown in Figure 21. Mehaffey [69] and Kong [71] considered a specific heat with a peak at 100-120°C because of the energy required to evaporate the water in the wood.

The specific heat can be calculated using Gammon's [74] equation:

$$C_p = (a + bT + 4.187u)/(1 + u) + \Delta c$$

Equation 5

where:

C_p (kJ/kgK): is the specific heat,

T (°C): is the temperature

u (kg/kg): is the moisture content.

a and b : are the coefficients that depend on wood species [73].

Δc : is the moisture correction used only by Janssens [5].

$$\Delta c = (23.55T - 1326u + 2417)u .$$

Figure 21-Specific heat of wood at elevated temperatures (Peng, 2010) [72]

2.3.3 Density

Wood density depends on the type of tree species and the moisture content present in the material. At 200°C the moisture evaporates causing a reduction in density.

As the temperature continues to rise, further decomposition occurs, which causes a further reduction of wood density. At 300°C, the wood is converted to char, and the density becomes nearly constant. Researchers [41] [53] [76] [77] found that the

reduction in density due to moisture evaporation is about 10% to 15% while the reduction in density due to charring is about 70% to 80%

Figure 22, produced by Peng [72], shows the relation between the density and temperature from three different studies.

Figure 22 - Density of wood at elevated temperatures (Peng) [72]

2.3.4 Modulus of Elasticity

The modulus of elasticity follows a similar trend as density. The modulus reduces by about 10% when the temperature increases up to 200°C, as the moisture present in the wood evaporates. The modulus is reduced steeply by about 90% due to the wood's thermal decomposition at 300°C when the wood turns to char. This trend of modulus of elasticity reduction was reported by several researchers [42] [43] [45] [75] [76], and summarized by Peng [72], as shown in Figure 23.

Kong [71], described a different reduction of the modulus of elasticity with temperature. He measured the reduction at 50% when the wood's temperature was elevated to 100°C. It was then reduced to 95% or greater as the temperature rose to 300°C. The reduction of the modulus of elasticity with temperature is the same for both parallel and perpendicular to grain directions [20].

Figure 23 - Modulus of elastic of wood at elevated temperatures (Peng) [72]

2.3.5 Tensile Strength

As expected, the tensile strength of wood decreases with the increase of temperature until it becomes negligible when wood becomes char. Researchers, [2] [73] [76], found that tensile strength follows a linear relation with temperature. Others predicted a reduction trend similar to that for the modulus of elasticity and density assuming a bilinear relationship. However, each researcher discovered a different breaking point relative to the increase in temperature. Thomas [73], found the point of tensile failure at 80°C. Konig and Walleij [71], found this point at 100°C and Schaffer

[45] at 200°C. At 300°C the wood is completely converted to char and loses all its tensile strength. The losses of tensile strength due to temperature of different studies were summarized by Peng [26] and are shown in Figure 24. Similar to the modulus of elasticity, the reduction in tensile strength is assumed to be the same in both directions parallel and perpendicular to the grain direction. (Laplanche, Dhima, & Rachet) [20], (Peng, Hadjisophocleous, Mehaffey & Mohammad) [78].

Figure 24 - Tensile strength of wood at elevated temperatures [26].

2.3.6 Compressive Strength

Researchers found that the reduction in the compressive strength of wood at elevated temperature, in both directions perpendicular and parallel to the grain, was similar to the reduction of the tensile strength. Knudson [2] and Janssens, [76] assumed a linear relationship up to 300°C. Others, such as Konig and Walleij, [71], Schaffer [45], and Thomas [73] assumed a bilinear relationship similar to the tensile strength. A summary of some of these researchers' test values are shown in Figure 25 reproduced from Peng [26].

Figure 25 - The compressive strength of wood at elevated temperatures (Peng) [26].

2.3.7 Shear Modulus

As reported by Peng [72], researchers proposed a similar bilinear relationship for the reduction of the shear modulus of wood at elevated temperatures. The shear modulus was reduced by about 40% at 100°C and there is a further rapid reduction until the wood loses all strength at 300°C as specified in Eurocode [66].

2.3.8 Shear Strength

The shear strength of wood was shown to have a similar reduction at elevated temperatures as the shear modulus reducing by about 50% at 80°C followed by a rapid loss up to 300°C [40].

2.4 Steel Properties at Elevated Temperature

Steel is an isotropic material whose behavior at elevated temperatures is less complex than wood due to the fact that it does not undergo thermal decomposition. Furthermore, its material remains consistent even at 300°C. Nevertheless, at these temperatures, the strength and load carrying capacity deteriorates. This can result in a higher rate of deflection and even failure. The degree of this deterioration depends on how much of its surface area is exposed to the elevated temperature as well as the number of sides exposed. Wider surfaces and thinner members cause more heat transfer to the section core.

In this study, the steel columns are protected against fire and the surface temperature is monitored to ensure it does not increase beyond the steel's threshold. This limit is the temperature at which the steel's load-bearing capacity is equal to its design load (Buchanan) [3]. The steel plates and bolts used to connect the steel columns to the wooden beam in an exposed connection are not protected, while the steel plate used in the concealed plate is partially protected by wood members and is exposed only from the bottom. The steel properties at elevated temperatures are well documented in Eurocode 3 [79].

2.4.1 Steel Thermal Conductivity

Steel thermal conductivity reduces linearly up to 800°C, then remains constant at about 27.3 W/m.K as shown in Figure 26, (Eurocode 3) [79]. The equations given in Eurocode for the calculation of thermal conductivity are:

$$\begin{aligned} k &= 54 - 0.0333T & 20^{\circ}C \leq T < 800^{\circ}C \\ k &= 27.3 & 800^{\circ}C \leq T < 1200^{\circ}C \end{aligned} \quad \text{Equation 6}$$

Figure 26 - Steel thermal conductivity variations with temperature (Eurocode 3) [79]

2.4.2 Steel Specific Heat

The change in steel specific heat with temperature is illustrated in Figure 27 as given in Eurocode 3, which shows four distinct areas. Eurocode gives the following equations to calculate the steel specific heat with temperature.

$$\begin{aligned}
 c_p &= 425 + 0.773T - 1.69 \times 10^{-3}T^2 + 2.22 \times 10^{-6}T^3 & 20^\circ C \leq T < 600^\circ C \\
 &= 666 + 13002/(738 - T) & 600^\circ C \leq T < 735^\circ C \\
 &= 545 + 17820/(T - 731) & 735^\circ C \leq T < 900^\circ C \\
 &= 650 & 900^\circ C \leq T < 1200^\circ C
 \end{aligned}$$

Figure 27- Steel specific heat variation with temperature (Eurocode 3) [79]

2.4.3 Density

Due to the fact that steel does not decompose at elevated temperatures, changes in its volume are minimal. Therefore, the density of steel (7850 kg/m³) remains constant.

2.4.4 Reduction Factor for Steel Mechanical Properties

The reduction of the steel Modulus of elasticity (E_T), effective yield strength ($f_{y,T}$) and the proportional limit ($f_{p,T}$) with temperature are shown in Figure 28 as given in Eurocode 3) [79].

Figure 28 - Mechanical properties of steel at elevated temperatures (Eurocode 3) [79]

The steel's yield strength is assumed constant up to 400°C then a steep reduction takes place up to 80% at 650°C followed by gradual reduction until the steel loses strength at 1200°C.

The steel modulus of elasticity and proportional limit start to reduce at 100°C. Three distinct stages of modulus of elasticity reduction occur. The first stage is between 100°C to 500°C. The modulus of elasticity is reduced by 40%. The following stage is between 500°C and 700°C which causes a steep reduction of about 85%. The final stage happens when the temperature is elevated between 700°C to 1200°C and the steel completely loses its yield strength.

2.4.5 Eurocode Equations for Mechanical Properties

Eurocode 3 [79] stress strain relationships for steel at elevated temperatures are shown below:

$$\begin{aligned}
\sigma &= \varepsilon E_T & \varepsilon &\leq \varepsilon_{p,T} \\
&= f_{p,T} - c + \left(\frac{b}{a}\right) \left[a^2 - (\varepsilon_{y,T} - \varepsilon)^2 \right]^{1/2} & \varepsilon_{p,T} < \varepsilon &\leq \varepsilon_{y,T} \\
&= f_{y,T} & \varepsilon_{y,T} < \varepsilon &\leq \varepsilon_{t,T} \\
&= f_{y,T} \frac{1 - (\varepsilon - \varepsilon_{t,T})}{(\varepsilon_{u,T} - \varepsilon_{t,T})} & \varepsilon_{t,T} < \varepsilon &\leq \varepsilon_{u,T} \\
&= 0 & \varepsilon &> \varepsilon_{u,T}
\end{aligned}$$

Equation 7

$$\begin{aligned}
\varepsilon_{p,T} &= f_{p,T} / E_T \\
\varepsilon_{y,T} &= 0.02 \\
\varepsilon_{t,T} &= 0.15 \\
\varepsilon_{u,T} &= 0.2 \\
a^2 &= (\varepsilon_{y,T} - \varepsilon_{p,T})(\varepsilon_{y,T} - \varepsilon_{p,T} + c / E_T) \\
b^2 &= c(\varepsilon_{y,T} - \varepsilon_{p,T})E_T + c^2 \\
c^2 &= (f_{y,T} - f_{p,T})^2 / [(\varepsilon_{y,T} - \varepsilon_{p,T})E_T - 2(f_{y,T} - f_{p,T})]
\end{aligned}$$

where:

$\varepsilon_{p,T}$: is the strain at the proportional limit,

$\varepsilon_{y,T}$: is the yield strain,

$\varepsilon_{t,T}$: is the limiting strain for yield strength and

$\varepsilon_{u,T}$: is the ultimate strain.

3 Experimental Research

The experimental part of this study consisted of several full-scale fire resistance tests. The test assembly was composed of a 1900 mm glulam wood beam connected to two steel columns (W shape) 3200 mm high.

3.1 The Furnace

The furnace used for the tests is located at the Full-scale Fire Research Facility of Carleton University in Almonte, Ontario. It was specifically designed to test the fire performance of connection assemblies. Figure 29 shows an exterior photo of the furnace and Figure 30 shows an interior photo focusing on one of the two propane burners. As seen in the figure, the furnace is equipped with a loading frame used to support the test assembly and to apply the load. Two blowers were installed on the burner's sidewalls, supplying fresh air for combustion. The furnace temperature was controlled in order to follow the standard fire (temperature) curve. Figure 31 is an interior view of the furnace showing the location of the shielded thermocouples and plate thermometer.

Figure 29- Exterior photo of the furnace

Figure 30- Interior photo of one of the two propane burners

Figure 31 -Shielded thermocouples and plate thermometers in the furnace

The interior dimensions of the furnace are 2.7 m by 2.7 m by 2.2 m high. The interior ceiling and walls were insulated with a layer of Fibrefrax® mineral blanket (25 mm thick).

3.2 Assembly Temperature

Figure 32 shows a cross section of the furnace with a test assembly in place indicating the location of loads, wood and steel thermocouple locations, while Figure 33 shows cross sections in the steel column and the wood beam indicating the number and location of the thermocouples used to record the temperature at different depths and locations.

Eleven thermocouples were used to record the temperature at the mid-section of the wooden beam at 20, 40 and at 70 mm from the surface. Eighteen thermocouples, nine at each connection, were used to record the temperature of the wood near each connection at different depths and elevations from the exposed surfaces. Six thermocouples, three at each connection, were used to record the temperature along the steel bolts' shank. Two thermocouples were used to record the temperature of the steel plates, one at each connection. Eight thermocouples, 4 at each column, were used to record the temperature of the column surface at the position where the plate is connected to the column.

Figure 32-Cross section of furnace with test assembly in place

Figure 33 - Cross section of steel beam and wood beam with the locations of the thermocouples.

A hydraulic cylinder, Model RD1610 from Enerpac®, mounted on the top steel beam, was used to apply the loads which were measured by a load cell (MTS® 661.22D-01) attached to the cylinder. The pre-calibrated load cell operated within the range of -250 to 250 kN.

Three linear variable displacement transducers (LVDTs) were used to record the displacement (curvature) of the beam specimen during tests. As shown in Figure 34, two LVDTs were mounted at the top, 300 mm from each connection, and one at the bottom of the beam at the center. The LVDTs were also pre-calibrated.

Figure 34 - Cross section of furnace with LVDT

In the furnace, six shielded thermocouples and two plate thermometers were installed, as shown in Figure 35. Shielded thermocouples were used to control the temperature of the furnace, in accordance with the CAN/ULC-S101 standard [37]. The shielded thermocouples were fabricated using No.18-gauge type K thermocouple wire insulated within a porcelain tube. The thermocouple with the porcelain tube was inserted into a 21 mm outer diameter (O.D.) steel pipe with a sealed end. The shielded thermocouples were placed at three levels (about 400 mm away) from the centerline of the furnace. In addition, metal-sheathed and mineral-insulated (MSMI) type K thermocouples (1.6 mm diameter) were used to measure temperatures in the wooden members by inserting them into pre-drilled pilot holes in the wood. MSMI type K thermocouples (0.8 mm diameter) thermocouples were used to measure temperatures along bolt shanks and the steel column section. (See Figure 33)

The furnace was also equipped with two plate thermometers at each connection. Plate thermometers act faster than shielded thermocouples and they are recognized as a direct measurement of the heat load experienced by a typical specimen in a fire test (Cooke) [80] (Wickstrom) [81]. Therefore, their measurements were used as boundary conditions in the numerical finite element modeling.

Three 5000-Series Ethernet IMP (from Solartron Mobrey Limited®) were set up as the data acquisition system. Each Ethernet IMP has a capability of 20 analog channels. The IMP devices were connected to a PC via an Ethernet cable and data was recorded at each second of the tests.

Figure 35- The locations of thermocouples and plate thermometer in the furnace

3.3 Furnace Temperature Verification

It was necessary to test the furnace temperatures prior to the fire-resistance tests in order to ensure it can follow the standard time-temperature curve. As described in CAN/ULC-S101 [37], results were obtained by averaging the data from the furnace thermocouple readings. The results of the fire tests lasting 1 hour or less were within $\pm 10\%$ under the standard time-temperature curve. Tests lasting between 1 and 2 hours were within $\pm 7.5\%$, and within $\pm 5\%$ for any tests exceeding 2 hours in duration.

Verification tests were performed using the furnace prior to testing any construction materials or assemblies. Propane flow rates were monitored and controlled in order to generate temperature curves within the acceptable tolerance. Figure 36 shows the

measured time-temperature curves of the six shielded thermocouples (TC1-6) and two plate thermometers (PT1 and PT2) from a typical verification test. The curves from the shielded thermocouples were very close to the standard curve during the verification test. The plate thermometer temperatures were higher than the shielded thermocouple temperatures, especially in the first 5-7 minutes, yet the difference became negligible after 10 minutes. The $\pm 10\%$ boundaries of the standard curve are also shown in Figure 36.

Figure 36 -Measured time-temperature curves and CAN/ULC-S101 standard curve without the wood specimen

It should also be noted, in fire-resistance tests, the heat generated by a combustible specimen contributes to the time-temperature curve. This extra heat is a factor, which must be considered and accounted for in order to follow the standard curve in a fire-resistance test. Fortunately, as shown in Figure 37, the measured time-temperature curves of the shielded thermocouples and plate thermometers from a typical fire test with the

specimen in place were within the limits of the standard. The heat release rate generated from the wood members was far less than the heat release rate generated from the propane burners; therefore, the heat contribution from the specimen to the temperature-time curve could be ignored (Peng, Hadjisophocleous, Mehaffey & Mohammad) [82].

Figure 37 - Measured time-temperature curves and CAN/ULC-S101 /ASTM E119 standard curve with a wood specimen

3.4 Description of Experiments and Test Specimens

A series of full-scale fire-resistance tests was conducted in the furnace (described above) to study the influence of various parameters governing the fire performance of hybrid connections between two steel columns and a glulam beam. The tests were done by exposing the test specimens to the standard time-temperature curve defined by CAN/ULC-S101 [37]. The test assembly consisted of two pin-pin steel columns (W150x37) CSA G40.21, 3200 mm long and a glulam beam (Nordic Lam 24F/ES12)

140x191x1900 mm, loaded using two point loads at one-third mid-section. The steel columns were fully protected while only the top surface of the wood beam was protected to imitate the presence of a floor/roof. A 3/8-shear tab plate with 4-A307 bolts was used to connect the beam to the steel frame. Figure 38 shows a specimen placed in the furnace and Figure 39 shows the assembly after the test.

Figure 38 - Specimen placed in furnace

Figure 39 - Specimen in the furnace after the test

The parameters considered for the tests include; the type of connection, the load ratio and the bolt diameter. 'Load ratio' is defined as the applied load during fire conditions, divided by the factored resistance of the test specimen. This resistance value is determined by the Canadian Standards Association, i.e. CAN/CSA O86-09 [36]. The load ratio applied on a structural member during a fire test has a significant effect on its fire resistance. According to CAN/ULC-S101 [37], it is recommended that the test specimen be subjected to nearly an identical load as the factored resistance of timber connections.

CAN/ULC-S101 also allows for loads of a lesser degree to be used during fire-resistance tests, provided the load conditions are identified and reported. The planned load ratios for each test specimen are shown in Table 1 and they range from 30% to 100%. Two different bolt diameters were used: 12.7 mm (1/2") and 19.1 mm (3/4"). The number of bolts was also changed from 2 to 4.

Three different types of connections were tested: the Concealed shear tab connection (CN), the Exposed shear tab connection (EX); and the Seated beam shear connection (SE). Figure 40 shows the steel plate corresponding to the three connection types.

Figure 40 -Steel plate connection types; A: Concealed B: Exposed C: Seated

3.4.1 Concealed Steel Plate

A steel plate 9.5 mm thick grade 300W was used to connect the glulam beam to the steel columns. A slot was cut in the wooden beam to accommodate the steel plate. Holes were drilled in the plate and the wooden beam to allow the bolts to connect the beam to the steel columns. The steel plate was welded to a base plate, which was then connected to the steel columns using 4 -10 mm A 307 bolts. This type of connection provides protection to the steel plate from direct fire exposure, but it may increase the heat transferred to the core of the wood through the steel embedded in the wood slots.

The Canadian code [36] requirements, related to the placement of the bolts in the joints, were satisfied such as e , e_Q , a , S_Q , and S_p , as shown in Figure 41, where:

e : is the unloaded edge distance $\geq 1.5 d$

e_Q : is the loaded edge distance $\geq 4d$

a : is the unloaded end distance $\geq 4d$ or 50 mm

S_Q : is the row spacing $\geq 3d$

S_p : is the spacing of fastener $\geq 3d$

d : is the diameter of the fastener

Figure 41 – CAN/CSA O86-09 bolt placement requirements in joints

3.4.2 Exposed Steel Plate

Two external steel plates 9.5 mm thick, grade 300W, were used to connect the glulam beam to the steel columns. No slot was cut in the glulam beam for this connection type. Holes were drilled into the plate and the wooden beam in order to attach the beam to the steel columns. The steel plates were welded on both sides of the base plate of identical thickness, and then connected to the steel columns using four 10 mm A307 bolts. Unlike the Concealed type, in this kind of connection the plate is directly exposed to the fire's heat flux and provides some protection to the wood from the fire and reduces heat transfer to the core of the wood.

3.4.3 Seated Bearing plate

This type of connection depends on the full bearing of the wooden beam on the steel plate. Although bolts are not needed to support the beam, common practice used in construction is to use bolts.

A 12 mm thick plate was used as the bearing plate and two 9.5 mm thick plates were welded to two opposite sides. This assembly was welded to the base plate and the plate was connected to the steel column using four 10 mm A307 bolts. The Seated connection is similar to the Exposed connection in that the plates are directly exposed to the heat flux. However it provides a better initial protection to the wood section from below due to the presence of the bearing plate. The Seated connection also reduces heat transfer to the core of the wood since only one bolt is used. Tests were also done without bolts.

3.5 Test Procedure

The test columns were connected to the steel loading structure that surrounds the furnace. Each column was restrained at its top and base against in-plane and out-of-plane lateral movements, yet they were allowed to shift rotationally to avoid the development of any unnecessary moments in the steel loading structure.

The test beam was inserted into the furnace through the furnace door, and then was bolted at both ends to the columns. The beam was then subjected to constant loading before and during the fire-resistance test. A hydraulic jack attached to a load cell was used to apply the vertical load over a vertically positioned steel post. A load-distributing bar was used to spread the weight between two points on the test beam, 700 mm apart. The load was maintained for a minimum of 20 minutes after which the test assembly was subjected to

gradually increasing temperatures that follow the CAN/ULC-S101-07 [37] standard time-temperature curve until failure.

3.6 Load Level

The magnitude of the load applied on a structural element during a fire test significantly affects the fire performance of the element and the connections. With a load less than the element's design capacity, the fire resistance increases and the need for high structural capacity decreases. If the load is close to the element's design capacity, the fire resistance is reduced. The ultimate capacity of structural elements must be higher than the design code's allowable capacity which is approximately 30% - 33% from the ultimate capacity [12] [83] [84].

During a fire, the occupancy load which represents a major factor of the live load in a typical building is reduced and it is an accepted assumption to consider that the applied loads are the dead loads and part of the live load (Buchanan) [3].

The load level recommended by the fire standards of CAN/ULC S101 [37], ASTM E119, [47] and ISO 834 [48] during fire tests, is the full capacity of the members in ambient conditions. However, if the level of the load applied during the test is lower than the design capacity value, it must be clearly identified and reported. (Buchanan) [3]

Therefore, there exists a high probability that the applied loads during a fire are less than 100% of the design loads at normal temperatures. During a fire, the occupancy load which represents a major factor of the live load in a typical building is reduced and it is an accepted fire design assumption to consider that the applied loads are the dead loads and part of the live loads.

Research found that an accepted level of load ratio during a fire is about 50% [3].

Eurocode – 5 recommends reducing the factored applied load for fire design by a factor of 0.6 [66]. For typical wood buildings in Canada, the reduced load factor for fire design should be about 0.41 of the factored load applied in ambient design. [1]. In this study load ratios of 30%, 60% and 100% were used in different tests.

3.7 Test Plan

A list of tests done with details and specimen dimensions is shown in Table 1.

The test plan consisted of 14 tests categorized into 6 groups.

Groups 1 and 2 were designed to cover the effect of load ratio on the fire resistance of the Concealed plate with 19.1 mm and 12.7 mm bolts connections respectively.

Groups 3 and 4 are similar to Groups 1 and 2 except they had the Exposed plate connections

Group 5 was a Seated plate connection with 12.7 mm bolts and without bolts and Group 6 covered the fire resistance of the Exposed and the Concealed plate connections with a 100% load ratio using 2- 12.7mm bolts.

Table 1- Experimental Tests Groups

Group	Test	Connection Type	Bolts diameter mm	Bolts #	Load Ratio	Serial #
1	1.1	Concealed	19.1	4	30%	1
	1.2	Concealed	19.1	4	100%	2
2	2.1	Concealed	12.7	4	30%	3
	2.2	Concealed	12.7	4	60%	4
	2.3	Concealed	12.7	4	100%	5
3	3.1	Exposed	12.7	4	60%	6
	3.2	Exposed	12.7	4	100%	7
4	4.1	Exposed	19.1	4	60%	8
	4.2	Exposed	19.1	4	100%	9
5	5.1	Seated	12.7	1	60	10
	5.2	Seated	12.7	1	100%	11
	5.3	Seated	No	No	60%	12
6	6.1	Concealed	12.7	2	100%	13
	6.2	Exposed	12.7	2	100%	14

4 Finite Element Modeling

A numerical heat transfer analysis was carried out for predicting the temperature distributions of timber connections in standard fire resistance tests. A structural model was also developed to predict the structural performance of the assembly. Both models were developed using the same approach utilized by Akotuah [9], who used the commercial finite element program ABAQUS /Standard (Hibitt) [39] that was validated using full-scale test results. Temperature-dependent properties of the test assembly, including thermal, strength and elastic modulus, were simulated along with brittle cracking failure of the wood.

4.1 Heat Transfer Model

To simulate the fire resistance test, the system was modeled using a sequentially-coupled thermal-stress procedure. This approach adopts three steps, first a transient heat transfer model is completed to determine the temperature distribution in the assembly, second a nonlinear static analysis is conducted to simulate the structural response of the specimen. Then, the heat from the fire is applied to the assembly while the load applied in the structural model remains constant.

Three-dimensional diffusive solid elements (DC3D8) in ABAQUS [39] were used to define every component of the assembly for the heat transfer model. Temperature-dependent thermal properties of conductivity, density and specific heat capacity were assigned to each element to determine heat conduction within the model assembly.

4.2 Heat Transfer Between Members

Heat transfer between discontinuous contacting surfaces of the assembly was simulated using the gap conductance feature, which is an interaction property used by ABAQUS to define the relation of contracting surfaces with the assembly. Heat transfer to the boundary surfaces of the assembly was modeled by supplying an appropriate heat flux to the exposed sides using a DFLUX user subroutine in ABAQUS. The heat flux to the boundary incorporated both the convective and radiation components of the heat from the furnace. The average time-temperature data recorded during the test (T_f) was used to estimate the flux to the boundary using Equation 8.

$$\dot{q}'' = h_c(T_f - T_s) + \emptyset \varepsilon_{ff} \sigma(T_f^4 - T_s^4) \quad \text{Equation 8}$$

where:

h_c = convective heat transfer

T_f = furnace temperature

T_s = surface temperature of the assembly

$\emptyset$ = configuration factor

ε_{ff} = effective emissivity

σ = Stefan – Boltzman constant, $5.67 \times 10^{-8} \text{W/m}^2\text{K}^4$

The full-Newton method in ABAQUS /Standard was used to solve the temperature at all points within the model. The temperature-time distribution within the assembly was applied as a predefined time-varying field in the subsequent structural model.

4.3 Structural Model

To simulate the structural response of the specimen, a nonlinear static analysis was conducted in two stages. First, the full load ratio was applied in increments. Then, the heat from the fire was applied to the assembly while the load from the first stage remained constant. Three-dimensional continuum elements (C3D8R) in ABAQUS were used to define all components of the assembly. Temperature-dependent elastic (isotropic and orthotropic) properties were assigned to the steel and wood parts respectively. One fourth of the assembly was modeled in ABAQUS due to the inherent symmetry of the assembly. See Figure 42. It should be noted that during a full scale test complete symmetry cannot be achieved due to the non-homogeneous properties of the materials used and some variation in the heat flux applied in the furnace. However, this is also the case if the whole assembly was modeled.

Figure 42 -One quarter Model in ABAQUS

4.4 Damage and Failure Model

Failure in the assembly was predominantly due to brittle crack (splitting) propagation in the glulam beam, which eventually led to the loss of load-carrying capacity. The extended Finite Element Method (XFEM) in Linear Elastic Fracture Mechanics (LEFM) [38] was used to model the formation and propagation of shear cracks in the continuum elements. Structural damage was initiated when the maximum shear stress criteria within any element was exceeded. Subsequent propagation of cracks is governed by fracture energies according to Griffith [85].

The initial application of the full vertical load ratio on the structural model did not lead to failure because the load applied was a fraction of the ultimate failure load. In the second part, the heat added to the assembly degraded the stiffness and strength of the materials, and caused charring. This led to an increase in internal stresses, damage and eventual failure of the assembly.

ABAQUS /Standard was able to track the progressive damage in the form of energy dissipated (ALLDMD). Figure 43 below is an example of a typical ALLDMD curve, which displays three distinct phases. The first was marked by a value of nearly zero, which indicated no inter-laminar damage (no cracks in the glulam). This was followed by a sudden rise in energy, which represents energy released in the formation of cracks and their propagation. The third phase had an almost constant peak value, which indicates that no more energy could be accommodated through damage. This signified that ultimate failure had occurred.

The ultimate time to failure of the model was observed at the point of sudden rise in the ALLDMD value as shown in Figure 43. This gave an indication of the time of failure for fire resistance.

Figure 43 Damage Dissipated Energy (ALLDMD) of the whole model [10]

4.5 Discretization

The first step in the finite element analysis approach was to break down the connection assembly into models, which could then be analyzed using ABAQUS . Each component of the assembly was created as a three-dimensional solid deformable part. Some parts such as the bolts, nuts and washers have many contact surfaces hence were lumped together to minimize the discontinuities while simulating the actual behavior of each component. Figure 44 to Figure 47 show the modeled assembly parts. The welded regions on the plates were also modeled as continuous and monolithic with adjoining sections. The manufactured glulam beam consisted of several lamellae of wood, which

was laminated along their grain to form one piece. The multiple lamellae of the glulam beam were modeled by partitioning the beam and applying material properties and material orientation to each layer (see Figure 44 to Figure 47).

Figure 44 -Steel column meshed model

Figure 45- Glulam beam mesh model

Figure 46 - Plate mesh model

Figure 47 - Bolt mesh model

Meshing of the connection assembly was performed locally on each component before the final assembly of the model. This was done to ensure the difference in geometry of the individual assembly components was accounted for while meshing. Curved geometrical sections were meshed so the smoothness of the surfaces was maintained.

This was accomplished by employing finer mesh sizes with smooth curvatures.

Fine meshes with smooth curvatures were also critical in discontinuous contact regions of the assembly such as the bolt and the bolt-hole areas where concentration and transfer of stress between individual elements were expected. To minimize the contact problems, which may arise during the simulation, ABAQUS required that the master (primary) surface had a coarser mesh than the slave (secondary) surface of any contact pair.

In Figure 44 to Figure 47, the bolt and bolt-hole regions were carefully knitted with finer mesh sizes to maintain the curvature of the geometry while slightly coarse mesh sizes were employed in other regions of the assembly. Mesh sizes were extremely important in obtaining an accurate solution to the problem within a reasonable amount of time. The appropriate mesh size for the analysis was chosen after an independent validation of both the heat transfer and structural models. The same mesh size, which conformed to both models, was selected and used in the final sequential coupled thermal-stress analysis. A similar mesh size facilitated the coupling of both analyses procedures during the model simulation.

4.6 Assembly and Contact Interactions

All the components of the assembly were created to enable the simulation of a quarter of the test assembly. The program was able to extrapolate from the equations, the behavior of the whole assembly. Figure 42 shows the modeled test assembly for simulation.

The assembly module feature in ABAQUS defines how the individual parts interact with one another. These interactions are areas of discontinuities where results are carefully transferred during the numerical simulation. Contact pairs were defined for interacting surfaces such as bolt-bolt hole, wood-steel plate and column-steel plate by assigning the “master” or “slave” roles to each of the contact pairs. The contact algorithm in ABAQUS requires that the master surfaces have coarser meshes than slave surfaces. Master surfaces are normally expected to play the major role in the interaction property of contact pairs because this is what defines the relation of contacting surfaces within the assembly.

The interaction property used in the thermal model was “gap conductance”. Gap conductance determines the conduction of heat from one contact element to the other. This value of the conductivity depends on the amount of clearance between the contacting surfaces. The other contact properties were the tangential and normal behavior of the contacting surfaces due to the mechanical behavior of the test assembly. The tangential behavior was defined by a friction coefficient of 0.3, which is typical of wood-metal surface interactions. The normal behavior between the contacting surfaces was defined as a “hard contact” in ABAQUS . A “hard contact” is based on the principle that stress can be transferred between surfaces at zero clearance and no stress is transferred when the surfaces are not in contact. All of the contact pairs and their respective contact properties were defined before the start of the numerical analysis in order to simulate the initial conditions of the assembled test specimen.

4.7 Loads and Boundary Conditions

Applicable loads and boundary conditions were defined on the assembly based on which type of analysis was being conducted. In the thermal model, the initial temperature of the assembly was included as a predefined boundary condition before the start of the analysis. The heat flux was defined as the thermal load on the assembly. The heat flux ($\dot{q}''$) included convective heat from the hot gases within the furnace and radiant heat from the heated internal furnace boundaries. This value of the heat flux can be estimated from Equation 8 by knowing the average recorded temperature (T_f) reaching the assembly at any point in time during fire exposure.

The temperature within the furnace varied over time and was regulated to follow the CAN/ULC –S101 time-temperature fire curve during the experiment. Figure 48 shows the average recorded temperature from a test, which lie within $\pm 10\%$ envelope of the standard time-temperature fire curve. The average recorded temperature was used as the input temperature (T_f) and ABAQUS estimated the corresponding heat flux load to the model assembly at every point in time during the simulation. The heat flux was applied uniformly over all the exposed surfaces of the test assembly during the analysis. Figure 49 also shows the assembled model with the applied heat flux load for numerical simulation.

Figure 48 - Recorded temperature within the furnace

Figure 49 - Applied heat flux load on the assembly

The mechanical behavior of the assembly was simulated by applying uniform pressure across two steel plates on the beam. An amount of pressure, equivalent to the corresponding load ratio on the assembly, was applied over 140 mm square plates during the analysis. The top and bottom of the steel column was pinned to simulate the hinge supports at the column ends during the experiment. The pinned boundary condition allowed for rotation in the plane of the assembly but restrained displacements in all three directions (X, Y, Z). Figure 50 shows the assembled model with the appropriate mechanical boundary conditions.

Figure 50 - Mechanical boundary condition of model assembly

4.8 Analysis

Simulating the fire resistance tests involved the integration of a fire model, heat transfer model and a mechanical model. The fire and heat transfer models incorporated the exposure of the assembly to the fire and the conduction of heat within the parts. The coupling of the heat effects from the fire and the mechanical load on the structure can be accomplished by a fully coupled thermal-stress or a sequentially coupled thermal-stress analysis procedure. The latter was chosen due to its simplicity in solving the coupled thermal-stress analysis of a system having several contact interactions. The sequential-coupled analysis procedure was done by first conducting a thermal analysis on the assembly to determine its transient temperature distribution during fire exposure. Once the temperature distribution of the assembly was successfully determined, the subsequent mechanical response was conducted in a separate structural analysis.

The numerical analysis in ABAQUS was conducted in a systematic procedure known as “analysis steps”. The default analysis step called “initial step” preceded any other included analysis step prescribed by the user. The initial temperature of the assembly was defined at this step for the thermal analysis, while contact interactions and pinned-end boundary conditions were specified in the initial step of the mechanical analysis.

In the thermal analysis, a pure transient heat transfer analysis was chosen as the analysis procedure. An initial temperature similar to the temperature measured in the tests was specified for the entire assembly prior to the start of the analysis. The heat flux on the exposed assembly boundaries was estimated from the boundary temperature (recorded from the tests) through a DFLUX subroutine and applied on the model assembly at each point in time. The DFLUX subroutine is a set of computer program instructions, which computes the distributed heat flux on a surface and transfers it to the simulation during analysis. Since the boundary temperature of the assembly varies with time, this results in a varying heat flux to the assembly. This heat flux is applied instantaneously on the assembly at any point in time and ABAQUS employs the full-Newton method in solving the three-dimensional transient heat conduction equation as defined by Equation 9.

$$\rho c \frac{\partial T}{\partial t} = \frac{\partial}{\partial x} \left(k \frac{\partial T}{\partial x} \right) + \frac{\partial}{\partial y} \left(k \frac{\partial T}{\partial y} \right) + \frac{\partial}{\partial z} \left(k \frac{\partial T}{\partial z} \right) + S \quad \text{Equation 9}$$

where, ρ = density (kg/m³) c = specific heat capacity (J/kg/K) t = time (s)

k = conductivity (W/m/K) S = source of heat (W/m³) and T = temperature (K)

The material properties of ρ , c and k were assigned to three-dimensional diffusive solid elements (DC3D8), which had been used to discretize the individual parts of the

assembly. These elements had eight independent nodes, each with a gauge able to describe the temperature at any point during the heat transfer analysis. The thermal properties of steel and wood materials were obtained from the literature and this provided the input for many of the variables in Equation 9 . ABAQUS conducted the heat transfer analysis in steps of the fire exposure time and an output time-varying temperature distribution on the assembly was obtained on the model assembly. The temperature distribution on the assembly was obtained for one hour of fire exposure and used in the subsequent mechanical analysis.

After a successful heat transfer analysis on the assembly was conducted, a thermal-stress coupled analysis was performed. This is a type of structural analysis, which incorporates the effects of temperature from the fire on the mechanical behavior of the structure. The structural elastic and strength properties of wood and steel specified in this instance were temperature-dependent and therefore varied as the heat from the fire penetrated the assembly over time. The fire resistance simulation analysis was conducted in two main steps.

The first stage was comprised of the incremental application of the structural load on the assembly until the full load ratio was reached. At this stage, the members of the assembly were stressed without reaching an ultimate failure of the connection assembly. In the second step of the analysis, the temperature results from the heat transfer analysis results were transferred into the assembly as a time varying “predefined field”. As the temperature increased, the elastic and strength properties of the assembly parts began to degrade over time while the applied load on the assembly remained constant. The elements became more flexible as the temperature increased and the corresponding

strength of wood and steel decreased, which gradually lead to greater susceptibility to damage and ultimate failure.

Damage in the connection assembly was preceded by an accumulation of small elastic deformations. Initial deformations occur when the full load ratio is applied on the assembly prior to exposure to fire. Subsequent deformations sustained in the assembly result from the flexibility of the material elements as their temperatures increase. Steel becomes softer and wood begins to char. Both effects reduce the elasticity of steel and wood elements at the connection. Steel fails after sustaining a large deformation (yielding) under a sustained applied stress. Charred wood also becomes more brittle and cracks can initiate and propagate. This propagation and accumulation result in the ultimate failure of the wood and eventually the entire assembly.

4.9 Finite Element Model Simulations

The finite element model simulations are shown in Table 2. The simulations consisted of 31 tests categorized into 11 groups to validate the numerical model and to study the parameters, which were not covered in the full-scale tests.

Group 1 was designed to cover the effect of 60% load ratio on the failure time of the Concealed plate with 4-19.1 mm bolt connections and to validate the model using experimental test results for the load ratios 30% and 100%.

Group 2 was designed to validate the model using experimental test results for the Concealed plate with 4-12.7 mm bolt connection at load ratios 30% and 60%.

Group 3 was designed to cover the effect of 30% load ratio on the fire resistance of the Exposed plate with 4-12.7 mm bolt connections and to validate the model using experimental test results for the load ratios 60% and 100%.

Group 4 was designed to cover the effect of 30% load ratio on the fire resistance of the Exposed plate with 4-19.1 mm bolt connections and to validate the model using experimental test results for the load ratios 60% and 100%.

Group 5 was designed to cover the effect of 30% load ratio on the fire resistance of the Seated plate with 1-12.7mm bolt connections, and to validate the model using experimental test results for the load ratios 60% and 100%.

Group 6 was designed to cover the effect of removing the bolts from the Seated connection for load ratios 60% and 100%.

Group 7 was designed to cover the effect of 30% and 60% load ratios on the fire resistance of the Concealed plate with 2-12.7mm bolt connection and to validate the model using experimental test result for 100% load ratio.

Group 8 was designed to cover the effect of 30% and 60% load ratios on the fire resistance of the Exposed plate with 2-12.7mm bolt connections and to validate the model using experimental test result for 100% load ratio.

Group 9 was designed to cover the effect of 30%, 60% and 100% load ratios on the fire resistance of the Concealed plate with 2-19.1mm bolt connection.

Group 10 was designed to cover the effect of 30%, 60% and 100% load ratios on the fire resistance of the Exposed plate with 2-19.1mm bolt connections.

Group 11 was designed to cover the effect of fixed loading (62.5 kN) (the minimum load corresponded to 100% load ratio for all connection types) the goal was to study the effect of the of connections type on the fire resistance.

The test results of Group 1, 4 and 5, were obtained by Ohene [9] [86].

Table 2 – Finite element modeling tests

Group	Test	Connection Type	Botls	Bolts #	Load Ratio	Load kN	Serial #
1	1.1*	Concealed	19.1	4	30%	21.5	1
	1.2*	Concealed	19.1	4	60%	43	2
	1.3*	Concealed	19.1	4	100%	71.5	3
2	2.1	Concealed	12.7	4	30%	21.5	4
	2.2	Concealed	12.7	4	60%	42.5	5
	2.3	Concealed	12.7	4	100%	71.5	6
3	3.1	Exposed	12.7	4	30%	18.75	7
	3.2	Exposed	12.7	4	60%	37.5	8
	3.3	Exposed	12.7	4	100%	62.5	9
4	4.1*	Exposed	19.1	4	30%	18.75	10
	4.2*	Exposed	19.1	4	60%	37.5	11
	4.3*	Exposed	19.1	4	100%	62.5	12
5	5.1*	Seated	12.7	1	30%	24	13
	5.2*	Seated	12.7	1	60%	48	14
	5.3*	Seated	12.7	1	100%	80	15
6	6.1	Seated	-	-	60%	48	16
	6.2	Seated	-	-	100%	80	17
7	7.1	Concealed	12.7	2	30%	10.65	18
	7.2	Concealed	12.7	2	60%	21.3	19
	7.3	Concealed	12.7	2	100%	35.5	20
8	8.1	Exposed	12.7	2	30%	14.5	21
	8.2	Exposed	12.7	2	60%	29	22
	8.3	Exposed	12.7	2	100%	48.5	23
9	9.1	Concealed	19.1	2	30%	20.7	24
	9.2	Concealed	19.1	2	60%	41.4	25
	9.3	Concealed	19.1	2	100%	69	26
10	10.1	Exposed	19.1	2	30%	18.6	27
	10.2	Exposed	19.1	2	60%	37.2	28
	10.3	Exposed	19.1	2	100%	62	29
11	11.1	Concealed	12.7	4	Minimum	62.5	30
	11.2	Seated	-	-	Minimum	62.5	31
*These tests were carried out by Ohene, (2014) [86] [9]							

5 Results and Discussion

5.1 Fire Performance Tests for the Concealed Plate Connections with 4 Bolts

Group 1 and Group 2

This section describes and discusses the results of the Concealed plate connections. The Concealed shear tab connected the beam to the column with the steel plate embedded in a center notch of the wood. The specific design details were described previously in Section 3.4.1. Figure 51 (a and b) shows a typical specimen in the furnace, before and after a test.

5.1.1 Furnace Temperature

It was necessary to calibrate the furnace temperatures prior to the fire-resistance tests in order to make sure the time-temperature curve followed the standard model; the flow of the propane gas in each test was continuously regulated to ensure the furnace temperature followed the standard curve temperature.

Test 1.1.

In Figure 52, the furnace temperature curves, from the 5 shielded thermocouples (TC) and the 4 plate thermometers (PT), are plotted for the Concealed 4-19.1-30% test. The temperature curves followed the standard curve quite well except for the initial 2-3 minutes, which showed a lower temperature than the standard curve. This was due to the large amount of heat the furnace surfaces absorb during this time. Immediately after failure at 33.4 minutes the burners were turned off, and this can be seen by the decrease of the temperature in Figure 52.

Figure 51 - Concealed specimen in the furnace before the test

Figure 52 - Furnace temperature vs. standard fire- CN 4-19.1-30%

Test 1.2

The furnace temperature curves for the Concealed 4-19.1-100% test were similar to Test 1.1. The average temperature curve followed the standard curve closely. Although, during the initial 3 - 4 minutes, the temperature was lower than the standard curve. The maximum temperature recorded ranged between 700-800C° due to the failure time of 13 minutes.

Figure 54, Figure 55 and Figure 56 show the furnace temperature vs. time for Test 2.1; Concealed 4-12.7 - 30%, Test 2.2; Concealed 4-12.7 - 60% - and Test 2.3; Concealed 4-12.7 - 100% respectively, the furnace temperature for all of these tests aligned well with the standard temperature curve. In Figure 55, the $\pm 10\%$ boundary was also shown as a typical example demonstrating the furnace temperatures were within the set boundaries. As stated in Section 3.3, the measured temperature curves may exceed the $\pm 10\%$ boundaries, as long as the area under the measured curves is within the $\pm 10\%$ of the area under the standard time-temperature curve.

Figure 53 – The furnace temperature vs. standard fire - CN 4-19.1-100%

Figure 54 – The furnace temperature vs. standard fire - CN 4-12.7.1 - 30%

Figure 55 – The furnace temperature vs. standard fire - CN 4-12.7 - 60%

Figure 56 – The furnace temperature vs. standard fire - CN 4-12.7-100%

5.1.1.1 Furnace Temperature Summary

Tests of the furnace temperature were recorded using 6 shielded thermocouples (TC) and 4 plate thermometers (PT). The flow of propane gas was regulated to ensure the average temperature followed the standard time – temperature curve. For every test, the time vs. temperature curves were in accord with the standard within the $\pm 10\%$ boundaries, except for the initial 3-4 minutes of the test.

5.1.2 Deflection

Test 1.1 Concealed with 4-19.1- 30%: Figure 57 depicts the beam deflection at 300 mm from each side, LVDT6 & LVDT7, and at the mid-span, LVDT5. Figure 58 shows the mid-span deflection over time. The initial deflection due to the applied load ratio of 30% was about 2.5 mm. When the fire load was applied, the deflection increased over time to 38 mm at failure. The applied load was maintained throughout the tests; however, the increase in deflection with time was due to the gradual degradation of the materials strength as a result of the fire exposure. The material became more flexible and weakened in relation to the increasing temperatures. After failure, the application of the load stopped and this is reflected in Figure 57 by the plateau at the end of the test.

It was also observed that a certain amount of deflection at both sides of the beam was recovered in the first 2-10 minutes of fire exposure. (See Figure 57) It is hypothesized that this was due to moisture migration from the wood and the subsequent contraction / expansion of the glulam layer due to different levels of heat exposure to the 3 sides of the beam. The top of the beam was protected and did not receive the same heat flux as the bottom and the sides of the beam. Along with the temperature increase, the wood

experienced thermal decomposition, which led to a reduction in modulus of elasticity, loss of stiffness and a drastic reduction in the beam's strength at its cross section.

Therefore the amount of deflection increased over time. After 20 minutes, one side of the beam started to show greater deflection than the other, as shown in Figure 57. Eventually, failure took place on this side. The recorded deflections at failure were 20 mm on one side and 10 mm on the other. The cause for this discrepancy can be attributed to the natural variability of wood temperature-dependence properties in addition to stress redistribution, which led to a concentration of stress on one side until failure.

Figure 57 - Beam deflection vs. time CN 4-19.1 - 30%

Figure 58 – The mid-span deflection vs. time CN 4-19.1 - 30%

Figure 59 shows the deflection along the beam length and Figure 60 shows the mid-span deflection over time for Test 1.2 - Concealed with 4-19.1 -100%. The initial deflection due to the applied load ratio of 100% was about 11 mm, which was increased to 40 mm at failure. A significant increase in mid-span deflecting was observed one minute prior to failure. The deflection nearly doubled from 22 mm at 12 minutes to 40 mm at 13 minutes. This was due to brittle failure at one side of the beam. A plateau of the deflection curve can be observed after failure.

Similar to Test 1.1, as shown in Figure 59, some deflection at both sides of the beam were recovered in the first 2-10 minutes of fire exposure; however, it was less significant than in Test 1.1 because the load ratio was higher for this test. This restricted the deflection recovery due to moisture migration and differential thermal expansion. The

deflection on the side where the failure eventually took place was 9 mm while only 4 mm was recorded on the other side.

Figure 59 - Beam deflection vs. time CN 4-19.1 - 100%

Figure 60 – The mid span deflection vs. time CN 4-19.1 - 100%

Test 2.1- Concealed with 4-12.7 -30%; Figure 61 shows the deflection along the beam's length while Figure 62 shows the deflection at beam mid-span for Test 2.1. The initial deflection, due to the applied load ratio of 30%, was about 2.5 mm, which was increased to 49 mm at failure. A rapid increase in the mid-span deflection from 32 mm at 39 minutes to 49 mm at 40 minutes was observed. As shown in Figure 61, some deflection at both sides of the beam was recovered in the first 15 minutes. It also shows a sudden increase in the deflection, recorded by LVDT7, from 3 mm at 20 minutes to 25 mm at 28 minutes. This did not change until failure because the ceramic rod used on this side broke at 20 minutes during the test. However the deflection on other side was recorded and shows a gradual increase in deflection up to 22 mm.

Figure 61 - Beam deflection vs. time CN 4-12.7- 30%

Figure 62 - Mid span deflection vs. time CN 4-12.7 - 30%

Test 2.2- Concealed with 4-12.7 - 60%; deflection along the beam's length is shown in Figure 63 and the deflection at the mid-span is shown in Figure 64. The initial deflection prior to starting the fire was about 6 mm and was gradually increased to 29 mm at 26 minutes. It then rapidly increased to 41 mm at 27 minutes. This clearly indicates that the formation of cracks started at the location of the connection and swiftly propagated to the mid-span resulting in a brittle failure. Figure 63 also shows the same trend of deflection recovery at the sides within the first 15 minutes. The recorded deflection on both sides at failure, were 12 and 22 mm; the higher deflection was on the side where failure took place. A plateau of the deflection curve was also observed after failure.

Figure 63 - Beam deflection vs. time CN 4-12.7 - 60%

Figure 64 – The mid-span deflection vs. time CN 4-12.7 - 60%

The beam deflection and the mid span deflection for Test 2.3- Concealed with 4-12.7 – 100% are shown in Figure 65 and Figure 66 respectively. The initial deflection prior to starting the fire was about 9 mm and was gradually increased to 25 mm at 17 minutes. The deflection then rapidly increased to 40 mm at 19 minutes. Figure 65 also shows the same recovery of deflection on the sides of the beam during the first 15 minutes. No significant difference in deflection was observed between the two sides. It increased gradually to 7 mm on one side and 10 mm on the other at failure. This was due to the similar degree of damage occurring on both sides of the beam at the connection point. This resulted in very similar deflections on each of the beam's sides. This result was contrary to other tests where the majority of the damage accumulated on one side of the beam. A plateau of the deflection curve was also observed after failure.

Figure 65 - Beam deflection vs. time CN 4-12.7 - 100%

Figure 66 - Mid span deflection vs. time CN 4-12.7 - 100%

5.1.2.1 Deflection Summary

Deflection at the mid-span and at 300 mm from each side of the beam was recorded. In the majority of the tests, a small recovery of the beam's deflection on both sides was observed in the initial 10 to 15 minutes. The preliminary deflection increased with the higher load ratio and ranged between 2.5 mm to 9 mm. (The effect of the load ratio on failure time will be discussed in a different section.) It was also noticed that the beams experienced a rapid increase in the deflection of their mid-span approximately one minute prior to failure due to the propagation of cracks emanating from the connection to the mid span. The mid-span deflection recorded at failure ranged between 38 mm to 49 mm.

5.1.3 Load Ratio

The load ratio is defined as the applied load in fire conditions divided by the factored resistance of the test specimen determined in accordance with the design standard published by the Canadian Standards Association, i.e. CAN/CSA O86-09 [36]. The applied load may be lower under fire conditions than the design loads adopted at ambient temperatures, because the majority of the occupancy loads are assumed absent during a fire event, Buchanan [3]. The load ratio applied on a structural member during a fire test has a significant effect on its fire resistance. According to CAN/ULC-S101 [37], it is recommended that the test specimen be subjected to nearly an identical load as the factored resistance of timber connections.

However, the fire standard of the CAN/CSAO86 [36] also allows for loads of a lower magnitude to be used during fire-resistance tests, provided the load conditions are identified and reported. During the tests, the hydraulic jacks were used to maintain the applied load ratio throughout the test.

Test 1.1 (CN-4-19.1-30%): Figure 67 plots the applied load ratio during the test until failure. The required load ratio of 30% was maintained throughout the test time within a boundary of -2 to +5%. A rapid decrease of the applied load ratio was observed at the time of failure (33.4 minutes). During failure the beam lost all of its strength and resistance to the applied load could not be recovered.

Test 1.2 (CN-4-19.1-100%): in Figure 68 shows the relation between the applied load ratio and time. The required load ratio of 100% was maintained throughout the test within the boundaries of -2 to +3%. It was observed that at 12.5 minutes the load ratio reduced

from 100% to 80%. At that time, the beam recovered the load resistance capacity and held a load ratio of about 90% before a rapid decrease in load ratio due to failure at 13 minutes. It is believed that cracks started at the connection and widened due to the applied load. The redistribution of stress occurred for about 0.5 minutes, which enabled the beam to recover some of its capacity until the cracks propagated to the mid-beam and caused the final failure.

Figure 67 – Load Ratio vs Time - CN-4-19.1- 30%

Figure 68 – Load Ratio vs Time - CN 4-19.1 - 100%

Test 2.1 (CN-4-12.7-30%): Figure 69 shows the relationship between the applied load ratio and time. The load ratio of 30% was maintained throughout the test within the boundaries of -2 to +5%. A sudden reduction in the applied load ratio was observed at the time of failure (40.27 minutes). This indicates the brittle failure of the beam due to the reduction in material strength and propagation of cracks in the wood.

Figure 69 - Load Ratio vs Time - CN 4-12.7-30%

Figure 70 shows the relation between the applied load ratio and time for test 2.2 (CN-4-12.7 - 60%): Similar to test 2.1, the required load ratio of 60% was maintained throughout the test within the boundaries of -2 to +3 %. The sudden reduction in the applied load ratio was observed at the failure time of 27 minutes.

For test 2.3 (CN-4-12.7-100%): the applied load ratio of 100% was maintained within the boundaries of +1 % to -4% as shown in Figure 71. It was observed that at 17 minutes, redistribution of stress took place within the structure for about one minute and load capacity was recovered from 88% to 92%. Final failure of the member occurred at 18.25 minutes.

Figure 70 – Load Ratio vs Time - CN 4-12.7 - 60%

Figure 71 – Load Ratio vs Time - CN 4-12.7-100%

5.1.3.1 Load ratio summary

The required load ratios of 30%, 60% and 100% were maintained during the test duration within the boundaries of -1% to +5%. At failure, a rapid reduction in the load capacity is observed. Load capacity recovery occurred for approximately 0.5 to 1 minute in Tests 1.2 and 2.3.

5.1.4 Specimen Temperature Profile

The penetration of heat into the structural members is a major factor in determining the performance of the structure assembly in fire conditions. To simulate the presence of a ceiling/floor the top of the beam was protected. The columns were also protected; however, due to the direct contact between the connection plate and the column, the steel plate conducted heat to the column. Therefore the temperature of the columns' surfaces at the connection, were monitored to ensure it did not increase beyond the steel's threshold. This section discusses the temperature distribution, within the columns, at the sections of the beam close to the connection points, and at the mid-span of the beam.

5.1.4.1 Steel temperature profile

Steel members, due to their high thermal conductivity during exposure to high temperatures experience significant reduction in stiffness and strength, as well as increased deformations. Steel yield strength decreases proportionally to an increase of temperature. However, its ultimate strength increases slightly at moderate temperatures before starting to decrease at higher temperatures [3].

5.1.4.1.1 Column temperature

Eight thermocouples, four at each column, were used to record the temperature of the surface of the columns at the position where the plate was connected to them. (See Figure 32 and Figure 33 in Section 3.2.)

Figure 72 shows the temperature profile of the columns over time. During Test 1.1, the columns' surface temperatures increased to a maximum of 180°C at C3 during 35 minutes of exposure near the connected steel plate. The figure shows the columns' surface temperatures at a distance of 300 mm from the center of the connected plate ranged between 80 to 140 °C. The steel's yield strength was assumed to be constant up to 400°C while the steel modulus of elasticity and proportional limit started to reduce at 100°C. (See Figure 28 Section 2.4.4) Therefore it is believed, due to the protection provided, the columns did not lose their yield capacity or stiffness even though the temperature at C3 in one test rose to 180°C , however the average temperature in the cross section did not exceed 100°C.

Figure 72 – The steel column's temperature vs. time -CN 4-19.1 - 30%

Figure 73 shows a similar heat profile in Test 1.2. However, the maximum temperature did not increase to more than 60°C, due to short time of exposure. The maximum temperature recorded at the time of failure (13 minutes) was about 45°C. However, the temperature increased to 60°C even after shutting the burners off, due to continuous conduction of heat within the column.

Figure 73 – The steel column's temperature vs. time -CN 4-19.1 - 100%

For test 2.1, the failure time exceeded 40 minutes. Therefore, the maximum temperature near the plate reached 145°C. The range of temperature near the plate was between 70°C to 120°C. (see Figure 74.)

At the load ratio of 60% the failure time was 27 minutes for Test 2.2. The profile of the column temperature in Figure 75 shows the effect of the lower exposure time on the steel column's temperature. The maximum temperature did not exceed 100°C at the time of failure. Again, when the time of exposure was reduced to 18 minutes for Test 2.3, the maximum temperature recorded was reduced to 65°C as shown in Figure 76.

Figure 74 – The steel column's temperature vs. time -CN 4-12.7 - 30%

Figure 75 – The steel column's temperature vs. time -CN 4-12.7 - 60%

Figure 76 – The steel column's temperature vs. time -CN 4-12.7 - 100%

5.1.4.1.2 Bolts and plate temperature

Six thermocouples, three at each connection, were used to record the temperature along the steel bolts shank. Two thermocouples were used to record the temperature of the steel plates, one at each connection. (See Figure 33 in Section 3.2.)

The measured temperatures in the bolts and plates at both side connections are shown in Figure 77 and Figure 78 for Test 1.1. The temperature gradient along the bolt shank was also measured. The temperature measured was slightly higher at T9, 20 mm from the wood face and at T11 at 60 mm from wood face compared to T10 at 40 mm from the wood face. This is most likely due to the heat conducted from the bolt heads and the plate increased the temperature at T9 and T11. However, T10 was in the center so the heat transfer from the bolt heads and the plate took longer. The bolts' temperature increased to

400°C after nearly 10 to 15 minutes of exposure and reached a maximum of 750°C to 800°C at the time of failure (33 minutes.)

Steel yield strength reduces linearly to 20% with an increase of temperature from 400°C to 650°C. However, significant yielding or bending was not observed on the 19.1 mm bolts. (See Figure 79) This may have been due to the small load ratio of 30%. In addition, the large bolt diameter increased the bolt stiffness.

The plate's temperature did not reach 400°C even at failure and no deflection or yielding was observed. (See Figure 79) This is because the plate was protected by the wood member and was exposed only from its base.

Figure 77 – The bolts & the plate temperature vs. time on the right side -CN 4-19.1- 30%

Figure 78 – The bolts and the plate temperature vs. time on the left side -CN 4-19.1 - 30%

Figure 79 – The steel plate and bolts for Test 1.1 -CN 4-19.1 - 30%

Similarly to Test 1.1, Test 1.2 had the same temperature trend for the bolts and plates.

However, due to the shorter time of exposure, failure occurred at 13 minutes at the

maximum temperature of about 380°C to 500°C for the bolts and 200°C for the plates.

Figure 80 and Figure 81 also show the temperature profile for the bolts and the plates at the left and right sides. T10 and T11 show almost identical temperatures, about 20-30% less than the temperature recorded by T9. Since T11 was 20 mm further down from the beam's face it received less heat conducted from the bolt heads, but the additional heat received from the plate compensated for this difference. However, due to shorter time of exposure, the heat conducted from the plate did not increase the temperature at T11 more than T10.

Significant yielding or bending was not observed on the 19.1 mm bolts. (See Figure 82)

The bolt was able to withstand the 100% load ratio without initial deflection and the short time of exposure did not degrade the steel's properties to the point where any significant reduction in steel yield strength or modulus of elasticity was reached.

Figure 80 – The bolts and plate temperature vs. time on the left side -CN 4-19.1 - 100%

Figure 81 – The bolts and plate temperature vs. time on the right side CN 4-19.1 -100%

Figure 82 – The bolts for Test 1.2 CN 4-19.1 100%

The temperature of the bolts and plates at both side connections were also measured for Group #2, the concealed plate with 4-12.7 mm bolts. Figure 83 and Figure 84 shows the temperature of the bolts and plates over time on both sides for Test 2.1. The temperature gradient along the bolt shank was found to be slightly higher at T9 and T11 than T10.

These results were similar to the other Concealed tests. The bolts' temperature increased

to 400°C after nearly 10 to 12 minutes of exposure and reached a maximum of 800°C to 900°C at the time of failure (40 minutes.) It was also observed that the temperature of the 12.7 mm bolts was higher than those in the test with 19.1 mm bolts at the same load ratio by about 6-10%. The heat transfers to the bolt through the exposed surfaces at both side of the beam as well as through the surface areas in contact with the steel plate. The section factor which is defined as the heated perimeter divided by the cross section area (H/A) is 0.21 m⁻¹ for the 19.1 mm bolts and 0.31 m⁻¹ for the 12.7 mm bolts. It is theorized that section with a large section factor will heat relatively slowly and sections with a small section factor heat rapidly, as expressed in Equation 10 below [87].

$$\Delta T_s = \frac{H}{A} \frac{\Delta t}{\rho c} \left[h_c (T_f - T_s) + \epsilon_{ff} \sigma (T_f^4 - T_s^4) \right] \quad \text{Equation 10}$$

Slight bending was observed in all the bolts as it can be noticed in Figure 85) The 12.7 mm bolts had slightly more bending than the 19.1 mm bolts, for the same load ratio. Plates at both sides did not show any signs of stress or bending.

Figure 83 – The bolts and plate temperature vs. time at right side -CN4-12.7 - 30%

Figure 84 – The bolts and plate temperature vs. time at the left side CN4-12.7 - 30%

Figure 85 – The bolts for Test 2.1 CN 4-12.7 30%

In Test 2.2, the load ratio was increased to 60% and the time of failure was 27 minutes. This was reflected in the temperature profile for the bolts and the plate. The maximum temperature of the bolts was 550°C to 590°C and for the plate was 200°C - 250°C. (See Figure 86 and Figure 87.) The bolts with a 60% load ratio experienced slightly more bending than those with a 30% load ratio. (See Figure 88)

Figure 86 – The bolts and plate temperature vs. time at the right side -CN4-12.7 - 60%

Figure 87 – The bolts and plate temperature vs. time at the left side-CN4-12.7 - 60%

Figure 88 -Bolts for Test 2.2 -CN 4-12.7 - 60%

For Test 2.3, the right side thermocouple T10 malfunctioned and did not provide any data during the test. (See Figure 89) T11 showed the high temperatures throughout the test in comparison with T9. The maximum temperature was 800°C on the right compared to 550°C on the left. This difference was irregular and it is believed that the thermocouples did not perform properly.

T11 on the left side was disconnected and did not provide any readings after 15 minutes. (See Figure 90) In general, the temperature profile of the bolts followed the same trend as in the other tests in this Group. The plate's temperature reached 300°C at the failure time of 18 minutes. No significant deflection was observed for the bolts even at 100% of the load ratio. (See Figure 91) Moreover, during the short time of heat exposure, the steel maintained its strength and stiffness because the temperature did not reach higher than 550°C.

Figure 89 – The bolts and plate temperature vs. time on the right side -CN4-12.7 - 100%

Figure 90 – The bolts and plate temp vs. time on the left side CN4-12.7 - 100%

Figure 91 – The bolts for Test 2.3 CN 4-12.7 = 100%

5.1.4.1.3 Summary of steel temperature

The steel members experienced different temperature conditions depending on the degree of exposure to heat and the configuration of the connection. The only area where heat could be conducted to the insulated columns was through their contact surface with the connection plates. At that location, the columns' temperature ranged between 60°C to 180°C. The average temperature at the columns' cross-section did not reach or surpass the steel's softening limit.

The steel plate was protected on both sides by the wooden member, as well as a mineral blanket on top. The only area the plates were exposed to high temperatures was from a slot at the base of the beam. The maximum temperature of the steel plates ranged between 200°C to 350°C. There was no deformation or sign of yielding observed on any of the plates.

The bolts experienced higher temperatures due to heat conduction between the bolts' heads and the steel plate to the bolts' shanks. The maximum temperature recorded for the

bolts ranged between 550°C -850°C depending on the amount of time they were exposed to the heat. A small degree of bending was observed for the 12.7 mm bolts. However, for the 19.1 mm bolts, the amount of bending was negligible.

5.1.4.2 Wood temperature profile

At elevated temperatures, wood suffers thermal decomposition and releases combustible vapors. This process continues until the wood chars at a temperature of about 280°C to 300°C (Buchanan, 2001) [3]. The char layer has no strength, but it insulates the core of the wood and reduces further heat transfer from the fire.

Twenty-nine thermocouples were used to record the temperature at the mid-section of the wooden beam, as well as both sides of the beam in proximity to each connection at different depths and elevations from the exposed surfaces. (See Figure 32 and Figure 33 Section 3.2.) The temperature distribution within the wooden beam is presented in the following section.

Figure 92 and Figure 93 show the temperature recorded near the connection. During Test 1.1, thermocouples 20 and 21 recorded lower temperatures than the other thermocouples, due to their depth of 40 mm from the surface. The temperature at this depth remained close to ambient temperature for the initial 10 minutes of the test and then rose to 100°C at failure. The increase in temperature can be attributed to heat conducted from the steel plate and the bolts to the connection area.

Figure 92 – Wood section temperature vs time near the left connection -CN 4-19.1- 30%

Figure 93 – Wood section temperature vs time near the right connection -CN 4-19.1 -30%

In general, all thermocouples located 20 mm from the beam surface recorded very high temperatures. Thermocouple 16, which was located at one corner at the base, experienced higher temperatures due to the two-dimensional heat transfer (this location receives heat from two directions, the side and the bottom of the beam). This corner also had a higher charring rate. The graph shows that the wood temperature exceeded 100°C after 4-6 minutes at 20 mm from the beam surfaces. At that level, water started to evaporate and moisture migrated through the wood section. The wood turned to char, at about 12-17 minutes when its temperature reached 300°C.

Figure 94 and Figure 95 present a graph of the temperature distribution along the beam's width and sides, at various times during its exposure to fire. They show that at a distance of 20 mm on each side of the beam, the temperature increased from ambient temperature to about 650°C. Failure occurred at 33 minutes. At temperatures between 280°C - 300°C the wood started to char and at 500°C, fissures and shrinkage cracks appeared in the char layer. These allowed the furnace's heat to directly affect the temperature recorded by the thermocouples. It was also observed that 1 to 2 minutes after the burner was turned off, the temperature on the two sides of the beam was reduced by about 100°C, while in the core, it rose by about 20°C. This was due to the heat migration from outer layer toward the core of the beam. However, the maximum temperature of the beam's core did not reach higher than 120°C.

Figure 94 – The temp distribution along the beam's width at right side connection - CN4-19.1- 30%

Figure 95 - Temp Distribution along the beam's width at connection - CN4-19.1 - 30%

The temperatures at the mid-span were lower than those at the connections. As presented in Figure 96, during the first 5 minutes, the temperature remained at ambient levels, and then started to increase. Thermocouples 29, 30, and 31 at the beam's core did not record temperatures greater than 50°C at failure. This was also true for point 33, which was located 20 mm from the top of the beam. The temperature at the beam's bottom corner, thermocouple 25, rose to 100°C at 10 minutes and 300°C at 22 minutes. The temperature recorded at failure, which occurred at 33 minutes, was 680°C.

Figure 96 – The wood's temperature vs. time at the mid-section - CN 4-19.1-30%

The temperature profile along the depth of the beam at the mid-span is presented in Figure 97. In the beam's core the temperature remained at ambient levels but, at the base of the beam (170 mm from the top), the temperature increased to 260°C. It should be noted that at 20 mm from the top, despite the presence of the insulation, the temperature

increased to 50°C. This indicated some leakage of heat through the protective layer at top of the beam. Unlike the depth profile, the width profile showed higher temperature on both sides of the beam. The maximum temperature at failure was 350°C for the top corner (Thermocouple 35) and 650°C for the bottom corner (Thermocouple 25). Relatively moderate temperatures, from 80°C to 200°C, were recorded at the mid points 26, 28 and 34. (See Figure 98)

Figure 97 – The temperature distribution along the beam’s depth vs. time CN 4-19.1 - 30%

Figure 98 –The temp distribution along the beam's width at the mid span - CN 4-19.1 - 30%

The temperature near the connections and at the beam's mid span for Test 2.1 is shown in Figure 99, Figure 100 and Figure 101. The temperature profile is similar to Test 1.1 due to the similar time of exposure to heat in both tests. The time of failure was 40 minutes. Figure 100 shows that after 10 minutes the temperature on the left top corner (point 24) started to show a higher temperature than point 16, at the bottom corner. This can be attributed to the partial failure of the glue between the top layers. This also occurred at the mid-span (see Figure 101). However, thermocouple 35 at the top corner, started to show a higher temperature than point 25 on bottom corner after 20 minutes. This indicates the glue failure propagated to the mid-span but it did not lead to the final failure. The beam failed due to group tear out and split of the wood at the connection point. (See Figure 102)

Figure 99 – The wood's temperature vs. time at the right side connection - CN 4-12.7- 30%

Figure 100 – The wood's temp vs. time at the left side connections - CN 4-12.7 - 30%

Figure 101 – The wood's temperature vs. time at the mid-span section - CN 4-12.7 - 30%

Figure 102 - Separation of the wood layer at the top of the beam & failure at the connection CN 4-12.7- 30%

At the mid-span a steep rise in temperature was observed after 20 minutes at the top and bottom of the beam as shown in Figure 101 and Figure 103. It is believed that major fissures emerged in the charred layer, which increased heat transfer to the top and bottom of the beam at a depth of 20 mm. This increase is also evident in Figure 104, which shows the temperature distribution within the beam depth at the mid-span.

Figure 103 – The temperature distribution along the beam's width vs. time -CN 4-12.7 - 30%

Figure 104 –The temperature distribution along the beam's depth vs. time -CN 4-12.7 - 30%

When the load ratio was increased to 100% in Test 2.3, the failure time reduced to 18 minutes. With less time of exposure the temperature of the wood did not exceed 400°C as it can be seen in the Figure 105 to Figure 107. The temperature profile for both sides and the mid-span of the beam are shown in Figure 105, Figure 106 and Figure 107 respectively. The maximum temperature recorded near the connections ranged between 375°C to 400°C at 20 mm from the beam's surface and 50°C at the beam's core.

Figure 105 – The wood's temperature vs. time at the left connection - CN 4-12.7 - 100%

Figure 106 – The wood's temperature vs. time at the right connection - CN 4-12.7 - 100%

Figure 107 – The wood's temperature vs. time at the mid-span section - CN 4-12.7 - 100%

The temperature profile within the beam's depth and width at the mid-span for Test 2.3 is shown in Figure 108 and Figure 109. The temperature remained at ambient levels (about 20°C) at Points 33, 32, 31 and 30. However, at Points 28 and 29 the temperature increased to 38°C. 20 mm from the base of the beam, at Point 27, the temperature increased to 180°C. This explains the shallow layer of char, 12 to 16 mm, observed in this test. The temperature at the top of the beam, Point 33, did not increase which indicates that the leak within layer of insulation was minimal. The temperature distribution along the width was much higher on both sides (190°C) in comparison to the core of the beam which did not exceed 25°C. (See Figure 109) Near the support, at 300 mm from beam edge the temperature was higher than at the mid-span due to the heat conduction through the plate at the connections. The maximum temperature recorded at

20 mm from the surface ranged from 280°C to 380°C. The beam's core never increased above 40°C. (See Figure 110 and Figure 111)

Figure 108 – The temperature distribution along the beam's depth vs. time CN 4-12.7 - 100%

Figure 109 – The temperature distribution along beam's width vs. time at mid-span CN 4-12.7- 100%

Figure 110 – The temperature distribution along the beam's width vs. time at the right connection CN 4-12.7- 100%

Figure 111 – The temperature distribution along beam's width vs. time at the left connection CN 4-12.7 - 100%

5.1.4.2.1 Summary of the wood's temperature

The temperature profile was measured at the mid-span and near the connections. The temperature was greater near the connections than the mid-span due to the heat conduction from the plate and bolts into the wood's core. Temperatures at 20 mm from the surface ranged between 180°C to 650°C. This was especially true at the bottom corners of the beam, due to the two-dimensional heat transfer received from the bottom and sides of the beam as opposed to only one dimension for other points. The wood's temperature at a depth of greater than 40 mm from surface ranged from 25°C to 100°C depending on the time of exposure. However, the core of the beam remained at the ambient temperature of around 25°C.

5.1.5 Charring Rate

The rate at which wood turns to char, called the charring rate, is measured in mm/min. Char occurs at temperatures between approximately 280°C – 300°C. Wood loses its strength and stiffness when it is turned to char and only the residual section (un-charred wood) has load bearing capacity. Therefore, determining the charring rate is an extremely important factor in fire safety design. To determine the charring rate at failure the burner was shut down and water was used to extinguish and cool the wood to ensure that the temperatures recorded by the thermocouples are less than 200°C to stop charring. Then the beam was left to reach ambient temperature. The residual width and depth of the wood was measured once the charred layer was removed. (See Figure 112, Figure 113 and Figure 114)

The charring rate is calculated by dividing the thickness of the charred layer by the time of exposure. The charred layer along the beam's width is calculated by finding the difference between the original width and the residual width. Then, that value is divided by two, due to the fact that the beam was exposed from both sides. The charred layer for the depth is calculated by finding the difference between the original depth and residual depth. This is because the beam is exposed at the base yet protected at the top. However, some heat leakage was observed at the connection site, which contributed in a small amount of char at the top layer. This in turn, increased the depth charring rate at the connection.

Table 3 and Figure 115 show the average charring rates at the mid-span and near the connection. The depth charring rate at the mid-span ranged from 0.57 to 0.69 and averaged 0.63 mm/min. The width charring rate ranged from 0.67 to 0.78 with an average of 0.72 mm/min. The difference between these two rates can be attributed to the fact that the beam width was exposed on two sides while the beam depth was exposed on one side. Nevertheless, the charring width at the connection averaged 0.67 mm/min for the depth and 0.76 mm / min for the width. This marginal difference was due to heat conduction by the plate and bolts into the core of the connection.

Figure 112- The residual width of the beam at the mid-span

Figure 113 – The residual depth of the beam at the mid-span

Figure 114 – The residual width at the connection

Table 3 – The charring rates for Group 1 and 2

Test #	Failure Time min	Charring at mid-span				Charring at connection			
		Depth mm	Rate mm/min	Width mm	Rate mm/min	Depth mm	Rate mm/min	Width mm	Rate mm/min
1.1	33.4	20	0.6	24	0.72	22	0.66	25.58	0.77
1.2	13	8.25	0.63	9.17	0.71	9.25	0.71	9.92	0.76
2.1	40.27	26	0.65	28	0.7	*		29	0.72
2.2	27	15.5	0.57	18	0.67	15	0.56	19.25	0.71
2.3	18.25	12.67	0.69	14.17	0.78	13.17	0.72	15	0.82
Average			0.63		0.72		0.67		0.76

*In this test splitting of wood was observed at both connections; therefore the depth measurement was not relevant.

Figure 115 – The charring rates at the mid-span and connection for Groups 1 & 2

5.1.6 Failure Time and Failure Mode

The governed failure mode, in ambient temperature design, for the Concealed shear tap connections was a yielding failure in Group 1 which used four 12.7 mm bolts and splitting failure for Group 2, which used four 19.1 mm bolts

In Test 1.1, the beam failed at 33.4 minutes as a result of wood splitting. On one side, the wood was torn out. Above the location of the bottom bolts; cracks emerged and partially propagated toward the beam's mid-span. (See Figure 116) At the other side, the wood separated underneath the bottom bolts. Severe elongation and charring around the bolts was observed. (See Figure 116-b) However, wood splitting at the top layer caused the final failure of the beam. (See Figure 116-a) No bending of the bolts or plates was noticed.

a) The group tear out and top cracks at the connection.

b) Elongation of holes and charring around the holes

Figure 116 - Failure at the connection Test 1.1
a) Group tear out; b) Elongation and charring at the bolt holes

The specimen of Test 1.2 sustained the fire load for only 13 minutes. Splitting of the wood started at the corner of the bottom holes and propagated to the mid-span resulting

in failure. As in Test 1.1, no bending of the steel components was observed and there was a minimum of elongation of the holes and of charring around the holes. (See Figure 117-b).

a) Bending of the bolts or plates was not observed for test 1.1

b) - Cracks started at the bottom holes - Test 1.2

Figure 117 - Failure at the connection; a) bolts and plates - Test 1.1; b) wood splitting - Test 1.2

Similar to the specimen in Test 1.1, the specimen in Test 2.1 failed due to wood splitting at one connection starting at the top bolts and partially propagated to the mid-span. The failure time was 40.27 minutes. Separation of wood section from top of the bottom bolts which also known as group tear out was also noticed on both sides along with severe elongation and some charring around the holes. (See Figure 118-a)

The specimen in Test 2.2 lasted for 27 minutes under fire exposure. Failure was characterized by splitting of the wood at the connection of one side at the bottom holes. There was group tear out underneath the bottom bolts. Splitting of the wood at the top holes also started but it did not propagate farther than the bottom crack that caused the failure. A separation of the laminated layer was also noticed on the same side. (See

Figure 118-b). Figure 119 (a and b) shows elongation and charring around the holes while Figure 120 shows the slight bending of the bolts.

- a) Splitting at top layer and group tear out - Test 2.1

- b) Separation of laminated layer - Test 2.2

Figure 118: a) Splitting of the wood - Test 2.1; b) Separation of the wood layers - Test 2.2

a) The separation in the wood layer - Test 2.2

b) The separation in the wood layer

Figure 119 - Test 2.2 a) The separation in the wood layer; b) The separation in the wood layer (close shot)

Figure 120 – The slight bending of the bolts - Test 2.2

5.1.6.1 Summary of time and failure mode

The failure mode and comments are summarized in Table 4 for all of the specimens in Groups 1 and 2. The predominant failure mode was splitting of the wood parallel to the grain at the connections. The cracks initiated at the edges of the bolt holes, either at the top or bottom. The bolts transferred the load through their shanks by bearing onto the wood fiber. This produced compression stress and led to the crushing of the wood. This occurs if the amount of pressure reaches or exceeds the plastic state and is considered to be ductile or yielding failure.

The compressed portion of the wood around the bolts produces lateral stress perpendicular to the grain. This leads to splitting when the stress exceeds the fiber's shear strength and is known as brittle failure. Pure brittle or pure ductile failure was not observed during these tests. Rather, the primary type of failure, which occurs during fires, is ductile. (Note the bolt's hole elongation.) As a result of the increased time of fire exposure and maintaining the applied load, the wood failed due to splitting.

During the 100% load ratio tests, the bottom splitting cracks propagated further toward the mid-span due to the high-tension stresses on the bottom layer. Group tear out was also noticed above and underneath the bottom bolts. At the lower load ratio of 30%, the time of exposure was extended to more than 33 minutes, which increased the heat transferred by the bolts and the plate to the core of the wood. This led to a separation of the wood underneath of the bottom bolts. However, this type of failure did not lead to final failure. That was caused by the redistribution of internal stress, which led to a concentration of the load at the top bolts and caused splitting at the top layer.

Table 4 - Time and Failure mode for Group 1 & 2

Test #	# of Bolts	Bolt diameter	Load ratio %	Failure time min	Failure mode
1.1	4	19.1	30	33.4	Splitting of wood started at one side at the bottom bolts. Group tear out at the bottom bolts on both sides, Elongation of the bolt holes No bending of the bolts Charring around the holes
1.2	4	19.1	100	13	Splitting of wood started at the bottom bolts on one side. Minimum elongation of the bolt holes. No bending of the bolts Minimal charring around the holes
2.1	4	12.7	30	40.27	Splitting of wood started at the top bolts on one side. Group tear out at the bottom bolts, Elongation of the bolts holes No bending of the bolts Charring around the holes
2.2	4	12.7	60	27	Splitting of wood started at the bottom bolts, small amount of splitting at the top bolts on one side Separation of the glulam layer at the connections Group tear out at the bottom bolts on one side, Elongation of the bolt holes. Minor bending of the bolts Minor charring around the holes
2.3	4	12.7	100	18.25	Splitting of wood started at the top bolts on one side Elongation of the bolt holes. Minor bending of the bolts

It was noticed that failure time increased with the decrease of the load ratio. Reducing the load ratio from 100% to 30% significantly increased the failure time by about 120% for the 12.7 mm bolts and 156% for the 19.1 mm bolts. The effect of load ratio on failure time will further discussed later.

No bending of the 19.1 mm bolts was observed due to their high level stiffness at the connection points. The 12.7 mm bolts experienced some minor bending at a load ratio higher than 30%.

5.1.7 Model Validation

As discussed previously in Section 4. 32 tests were modeled using ABAQUS. They were designed and categorized into groups for the purpose of validating the model by using the full-scale test results as well as to study other parameters not covered by the full-scale tests.

Table 5 lists the models for Groups 1 and 2 for Concealed shear tab connections with the failure times and modes.

Table 5 - Model test for Groups 1 and 2

Group	Test	Connection Type	Bolts Dia mm	Bolts #	Load Ratio	Failure time min	Failure mode
1	1.1*	Concealed	19.1	4	30%	34.7	Wood splitting
	1.2*	Concealed	19.1	4	60%	19	Wood splitting
	1.3*	Concealed	19.1	4	100%	13	Wood splitting
2	2.1	Concealed	12.7	4	30%	45.6	Combination of wood splitting and bolts yielding
	2.2	Concealed	12.7	4	60%	30.2	Combination of wood splitting and bolts yielding
	2.3	Concealed	12.7	4	100	20	Combination of wood splitting and bolts yielding
* These numerical tests were carried out by Ohene (2014) [86] [9]							

5.1.7.1 Thermal model

The thermal model was validated by comparing the temperature values recorded by the model, with the recorded temperature at relevant points during the full-scale tests. Group 1 was validated by Ohene, (2014) [86] who concluded that the model generally is able to accurately predict the temperature of the bolts and wood within a range of $\pm 11\%$. The model predicted the temperature of the wood sections within a range of about $\pm 9\%$ when compared to the full-scale tests.

The results of Test 2.2 were used to validate the model for Group 2. The temperature during fire exposure at Points T11 and T12 at the bolts' shank and plates respectively, were compared to the temperature predicted by the model as shown in Figure 121 and

Figure 122. The model predicted the temperature within ± 12 -14%. The predicted temperature at the plate was lower in the model than in the full-scale test. This can be attributed to the fact that the top of the beam in the model was programmed to be protected by insulation with no heat transfer to the plate from the top, while it was found in the full-scale test, there was some heat transfer, which resulted in higher temperatures at the plate.

Figure 121 - Bolt temperature vs. time - T11

Figure 122 - Plate temperature vs. time - T12

The temperature development in the wood section at Points 16, 25 and 28 as predicted by the model and the as recorded in the full-scale test are shown in Figure 123. There is good agreement between the results of the test and the model. (See Figure 123) The model overestimated the temperature after 20 minutes of fire exposure and also predicted a decrease in the temperature toward the beam's core. T16, which is 20 mm from the beam's surface, shows a higher temperature than T28, which is located 70 mm from the beam's surface.

(a) T16

(b) T25

(c) T28

Figure 123 – The wood section's temp vs. time - a) T16, b) T25, c) T28

The increase in temperature due to fire exposure and the charring layer thickness were also compared. This was in order to validate the reliability of the input data used to represent the materials' properties used in the model. Figure 124 is a typical temperature contour as predicted by the model. The temperature along the width and the depth of the beam are in agreement with the full-scale test results as shown in Figure 125 and Figure 126.

Figure 124 - Temperature distribution in the model at 27 minutes

Figure 125 - Temperature distribution at the beam's width full scale test– Test 2.2

Figure 126 - Temperature distribution at the beam's depth – Test 2.2

The temperature at 20 mm from both sides of the beam were within the range of 400° - 500°C and within 22 - 80°C at the beam's core after 27 minutes of exposure. (See Figure 125) A similar range is presented by the model, in green and blue contour colors. However, the model did not predict temperatures higher than ambient conditions at the top of the beam. During the full-scale test, the temperature at the top of the beam increased to 60°C (see Figure 126), due to the small heat leak through the insulation layer. In the model, the beam was perfectly protected at the top resulting in no increase in temperature.

The depth at which the wood temperature exceeds 280°C and turns to char is represented by the light blue to red contour colors. These were measured to calculate the thickness of the charring layer predicted in the model. As shown in Figure 124, the char layer was 23

mm on the side and bottom of the beam compared to 18 mm and 15.5 mm measurements from the test. (See Table 3). The asymmetrical char layer in the test is a result of the uneven heat flux distribution, while the heat flux in the model was uniformly distributed to all exposed surfaces. The failure time predicted by the model for Test 2.2 is 30.2 minutes. Accordingly, the charring rate was 0.76 mm/min. While the average charring rate measured by the test was 0.72, the model predicted the charring layer within $\pm 5\%$. The model also predicted the penetration of heat through the bolts and plate to the core of wood as well as the char around the bolts as seen in Figure 127.

Figure 127 - Temperature distribution due to the metal components at the Concealed connection

5.1.7.2 Failure time

In the model, the specimen was subjected to a load ratio that was maintained at a constant rate throughout the simulation. It was then followed by the fire load applying the transient temperature solution on the assembly. The added heat degraded the strength of the materials gradually, and led to increased stress and damage, and finally failure. The estimated failure time was recorded when the model indicated sudden splitting of the wood. This is marked by the sudden rise in the curve of energy dissipated by damage (ALLDMA) with time shown in Figure 128 as discussed previously in Section 4 and reported by Ohene [10], [86]

Figure 128 – The progressive damage of the Concealed connection [10] [86]

However, when the failure mode is a combination of wood cracking and bolts yielding, the estimated failure time is recorded by following the time in which the cracking was generated and propagated. (See Figure 129) It is also necessary to track the strain on the

bolts if it exceeds the steel's yielding strain. The range of strains that exceeds the steel's strain are colored from light blue to red as shown in Figure 130.

Figure 129 – The cracks generated at bolts' edge

Figure 130 - Strains at the bolts exceeded the steel's yield strain at 30.2 minutes

Table 6 lists the time of failure for the Concealed connections tests in Group 1 and 2 from the model and the tests. These are also illustrated in Figure 131. The model predicted the time of failure with variation of 0 to 13.24% . The model was more accurate when failure was due to splitting, Ohene [86], and the variation did not exceed 3.89%. This value increased and ranged between 9.59 to 13.24% when the mode of failure was a

combination of wood splitting and bolts yielding. This is due to the fact that there was no variable used by the model to simulate the plasticity of wood.

Table 6- Test and model time of failure

Group	Test	Connection Type	Load Ratio	Failure time min		Variation
				Model	Test	
1	1.1*	Concealed	30%	34.7	33.4	3.89 %
	1.2*	Concealed	60%	19	-	
	1.3*	Concealed	100%	13	13	0%
2	2.1	Concealed	30%	45.6	40.27	13.24%
	2.2	Concealed	60%	30.2	27	11.85%
	2.3	Concealed	100%	20	18.25	9.59%

Figure 131 – Failure time of CN Connection; Test vs. Model

5.1.8 Discussion

In this section the effect of different parameters on the fire resistance of the Concealed connections will be discussed.

5.1.8.1 Load ratio effect

The relation between the load ratios and failure times for Groups 1 and 2 are shown in Figure 132. The failure time from the model was considered when there was no full-scale test data available. The failure time for each test is shown as symbols while the line represents the possible power relation for each group with the power formula, also included. The specimen with a load ratio of 30% had a higher failure time. For Group 1, the failure time increased from 13 to 33.4 minutes (157%) by reducing the load ratio from 100% to 30%. For Group 2, the increase of failure time was about 22 minutes (120%) for the same reduction in load ratio. Reducing the load ratio from 100% to 60% increased the failure time by 6 minutes (46%) for Group 1 and 8.75 minutes (48%) for Group 2. Further reduction of the load ratio from 60% to 30% improved the failure time by 75% for Group 1 and 49% for Group 2.

Figure 132 – Failure Time of the Concealed Connection for Groups 1 and 2

The effect of the load ratio on the mid span deflection is seen in Figure 133 and Figure 134 for Groups 1 and 2 respectively. Higher load ratios led to an increased initial deflection at the mid-span. In both groups, initial deflection ranged between 2.5 mm to 10 mm. At a 100% load ratio, the increase of deflection prior to failure was more pronounced in comparison with load ratios of 30% and 60%, due to high tension stress on the bottom fibers, which led to rapid increase of deflection prior to failure. The final deflection prior to failure was about 40 mm for all tests except for Test 2.1 (CN- 4-12.7 - 30%) which had a deflection of 49 mm.

Figure 133 – The mid-span deflection vs. time Group 1

Figure 134 – The mid-span deflection vs. time Group 2

5.1.8.2 Bolts diameter effect

The role played by the bolt's diameter can be significantly depending on the type of load and connection configuration. In Groups 1 and 2, four bolts were used with a diameter of 19.1 mm and 12.7 mm respectively. Figure 135 represents the effect of the bolt's diameter. Along with the load ratios, all other parameters remained the same for both groups. The figure shows that by reducing the bolt's diameter the failure time increased. The failure time improved by 6.87 minutes for a load ratio of 30%, 8 minutes for 60%, and 5.25 minutes for 100%. Increasing the bolt diameter from 12.7 mm to 19.1 mm reduced the failure time by 20 – 40%. This is because the greater diameter of the bolt increased the area in contact with the wood. Therefore, heat was transferred to a larger area in the core of the wood.

Figure 135 – The effect of the bolt diameter on the failure time for Groups 1 and 2

The effect of the bolt's diameter on the mid-span's deflection can be seen in Figure 136. At a load ratio of 30% there was no difference in the mid-span deflection in the first 13 minutes between the 12.7 mm and 19.1 mm bolts. However, with continued exposure, the connection with the 12.7 mm bolts showed a lower deflection by about 50%. On the other hand, at a higher load ratio, 100%, the connection with 12.7 mm bolts showed a lower mid-span deflection by about 10%. From the start of the test, both curves were nearly parallel.

Figure 136- mid span deflection vs. time for Groups 1 and 2

5.2 Tests with Exposed Plate Connections with 4 Bolts Group 3 and Group 4

This section describes and discusses the results of the Exposed plate connections. The Exposed shear tab connects the beam to the column with the two steel plates at each side of the beam; details were described previously in Section 3.4.2. Unlike the

Concealed connections the Exposed plate is directly exposed to heat flux and provide partial protection to the wood behind at the initial stage of heat exposure. The degree of protection depended on the plate's thickness, and the amount of water vapor generated from the wood moisture at 100 °C. Figure 137 (a and b) shows a typical specimen in the furnace, before and after the test.

a) Before the test

b) After the test

Figure 137 – The Exposed connection specimen in the furnace before and after the test

5.2.1 Furnace Temperature

Similar to the Concealed connections, the furnace temperature curves from the 5 shielded thermocouples (TC) and the 4 plate thermometers (PT) were plotted for the Exposed connections tests. The temperature curves followed the standard curve quite well except for the first 2-3 minutes when the temperatures were lower than the standard curve. Figure 138 for Test 3.2 and Figure 139 for Test 4.1 are typical curves for the Exposed shear tab connection tests.

Figure 138 - The furnace temperature vs time- EX 4-12.7-100% Test 3.2

Figure 139 – The furnace temperature vs time- EX 4-19.1-60% Test 4.1

5.2.2 Deflection

Deflection along the beam length for Test 3.1, EX-4-12.7-60%, is shown in Figure 140, and the mid-span deflection vs. time is plotted in Figure 141. The initial deflection at mid-span due to the applied load ratio of 60% was about 6.0 mm. This increased during the fire exposure to 65 mm at failure. Similar to the Concealed shear connections a plateau of the deflection curve after failure can be observed in Figure 141. The same phenomena of deflection recovery at both sides of the beam were observed during the first 2-10 minutes of fire exposure as shown in Figure 140

Figure 140 –The beam deflection vs. time EX 4-12.7 60% - Test 3.1

Figure 141 The beam's mid-span deflection vs. time EX 4-12.7 60% - Test 3.1

When the load ratio increased to 100% in Test 3.2, the initial mid span deflection increased to 10 mm while the mid span deflection increased to 49 mm just prior to failure and suddenly increased to 59 mm at failure at 22.15 minutes. (See Figure 142 and Figure 143) Deflection recovery at the beam's ends was observed in Test 3.2 similar to Test 3.1; however, it was less pronounced due to the higher applied load ratio.

In Test 4.1, with bolt diameter of 19.1 mm and load ratio of 30%, the beam behaved in a similar manner to Test 3.1 with an initial mid-span deflection of about 5 mm, and 50 mm deflection prior to failure. At failure the deflection increased to 65 mm. (See Figure 144 and Figure 145)

Figure 142 – The beam deflection vs. time EX 4-12.7 100% - Test 3.2

Figure 143 – The beam's mid-span deflection vs. time EX 4-12.7 100% - Test 3.2

Figure 144 –The beam's mid-span deflection vs. time EX 4-19.1 60% - Test 4.1

Figure 145 – The beam's deflection vs. time EX 4-19.1 60% - Test 4.1

The load ratio increased to 100% in Test 4.2. The beam in this test had an initial mid-span deflection of 7 mm and gradually increased to 20 mm at 15 minutes followed by rapid increase to 34 mm at the failure time of 18 minutes. Failure was due to wood splitting at one end of the connection. (See Figure 146.) Unsymmetrical deflection was observed at both ends of the beam as shown in Figure 147. The side that eventually failed had a higher deflection by about 60% than the other side, which is caused by the formation of a crack at one side of the beam that led to final splitting failure.

Figure 146 –The beam's mid-span deflection vs. time EX 4-19.1 100% - Test 4.2

Figure 147 – The beam's mid-span deflection vs. time EX 4-19.1 100% - Test 4.2

5.2.2.1 Deflection Summary

The deflections at the mid-span and at both sides of the beam were recorded. Similar to the Concealed connections, the beams with the Exposed connection also showed a small recovery of beam deflection at both sides for the first 10 to 15 minutes. The initial deflection increased with the increases of the load ratio and ranged between 5 to 10 mm. It was also noticed that the beams experienced rapid increase in the mid-span deflection one minute prior to failure due to the propagation of cracks from the connection to the mid-span. The mid-span deflection recorded at failure ranged between 30 to 65 mm.

5.2.3 Load Ratio

Similar to Groups 1 and 2, the applied load ratio vs. time was also plotted for all Tests in Groups 3 and 4. Typical graphs for Tests 3.1 and 3.2 are shown in Figure 148 and Figure 149 respectively. The required load ratio of 60% and 100% was fairly maintained throughout the test within a boundary of ± 1 to 4%. A rapid decrease of the applied load ratio was observed at failure time, because at failure, the beam lost all the strength and resistance to the applied load could not be recovered.

Figure 148 – Load Ratio vs Time - EX-4-12.7-60%, Test 3.1

Figure 149 - Load Ratio vs Time - EX-4-12.7-100%, Test 3. 2

5.2.4 Specimen Temperature Profile

In this section, the temperature distribution of the columns, plate, bolts shank and of the wooden beam at sections close to the connection and at the mid-span, are presented.

5.2.4.1 Steel Temperature Profile

The temperature of the steel columns, plate and bolts are presented in this section.

5.2.4.1.1 Columns Temperature

Eight thermocouples, 4 at each column, were used to record the temperature of the column surface where the plate connects to the column, see Figure 32 and Figure 33, Section 3.2. Figure 150 shows a typical plot of the columns temperature vs. time for Group 3 tests and Figure 151 as a typical curve for Group 4 tests. Both curves clearly indicate that due to the protection provided to the columns, the temperature did not exceed 100°C at the columns section located away from the plate and ranged between 110 to 220°C near the plate.

Figure 150 – The steel column's temperature vs. time EX 4-12.7-60%

Figure 151 – The steel column's temperature vs. time EX 4-19.1-100%

5.2.4.1.2 Temperature of bolts and plate

The temperature profile of the bolts shank and plate temperature is shown in Figure 152 and Figure 153 for Tests 3.1 and 3.2. The recorded temperature along the bolt shank, as expected, shows higher temperatures at the point nearest to the face of the beam, T9, than the point near to the core of the beam T11. The temperature recorded on the exposed plate was even higher.

The temperature at the bolts after 10 min of exposure ranged between 250°C to 300°C. The maximum temperature at failure at point T9 for Test 3.1 was about 800°C and the maximum plate temperature was 900°C. In Test 3.2, the temperature recorded at Point T9 was 600°C while the plate temperature was about 820°C. Unlike the Concealed plate connections, the Exposed plate did not transfer additional heat to the core of the beam and it did not increase the temperature at T11, which showed temperatures slightly less than those recorded at T10.

Figure 152 – The bolts and plate temperature vs. time EX 4-12.7-60% - Test 3.1

Figure 153 – The bolts and plate temperature vs. time EX 4-12.7-100% - Test 3.2

Similar to Test 3.1, Test 4.1 and Test 4.2 had similar temperature profiles for the bolts and plates. However, due to lower time of exposure, failure time was 26 min and 18 min for the load ratios of 60% and 100% respectively. The maximum temperature was about 550 to 700°C for the bolts and 700 to 800°C for the plates, as shown in Figure 154 and Figure 155. It was also observed that the bolts temperature near the bolts head (T9) in the case of the Exposed connections was higher than those for the Concealed connections by 16% to 30%. This was due to the extra heat conducted through the interface between the exposed steel plate and the bolts shank at the face of the beam.

Figure 154 – The bolts and plate temperature vs. time EX 4-19.1-60%- Test 4.1

Figure 155 – The bolts and plate temperature vs. time EX 4-19.1-100% - Test 4.2

It was also noticed that smaller size bolts, 12.7 mm, had significant deformation at 60% load ratio and a long exposure time, 38.6 min, see Figure 156 (a). Bending of bolts was less significant in Test 3.2 even though the load ratio was increased to 100%. This may be due to the shorter time of exposure of 22.15 min, see Figure 156 (b). On the other hand, for Test 4.1 and 4.2 larger size bolts, 19.1 mm, were used and no bending was observed. (See Figure 157.)

a) Steel plate and bolts EX 4-12.17 60%- Test 3.1

b) Bolts bending EX 4-12.17 100%- Test 3.2

Figure 156 - Bolts bending a) test 3.1, b) test 3.2

Figure 157 – The steel plate and bolts EX 4-19.1 60% Test 4.1

5.2.4.1.3 Summary of steel temperature

Similar to the Concealed connections, the steel members had different temperature profiles depending on the degree of exposure. The temperature at the vicinity of the plate ranged between 100°C to 250°C, which was higher than those recorded for the Concealed connections, because the unprotected plate conducted more heat to the column through the interface between the columns and the plate. However, the average temperature in the column cross-section did not reach the limit at which steel starts to lose strength and stiffness.

Unlike the Concealed connections, the plate in the Exposed connections was not protected therefore the maximum plate temperature increased to 900°C; however, no deformations were observed on the plates. On the other hand, the bolts had higher temperatures than those in the Concealed connection near the bolt's head due to the higher heat conducted from the interface area between the bolt's shank and the exposed plate. The maximum temperature recorded for the bolts ranged between 450°C - 800°C depending on exposure time. Significant bending of the 12.7 mm bolts was observed; however, for the 19.1 mm bolts the bending was negligible.

5.2.4.2 The wood's temperature profile

Figure 158 shows the temperature development of the beam near the connection for Test 3.1. Points 20 and 21 recorded lower temperatures than other points because these points are at a depth of 40 mm from the surface. The temperature at this depth remained at the initial temperature for the first 20 minutes and rose to 100°C, at failure. Similar to the Concealed connections, points that are at a distance of 20 mm from beam

surface recorded very high temperatures. Point 16, which was located at the bottom corner of the beam, recorded higher temperatures due to the two –dimensional heat transfer from bottom and side of the beam. The wood temperature at these points increased to 100°C, after 5-7 min and reached the char temperature of 300°C at about 15-20 min.

Figure 158 – The wood's temperature vs. time at the connections Test 3.1

The temperature distribution along the beam width at different times during the test is shown in Figure 159. The temperatures rose to about 600°C at 20 mm from the face of the beam and remained within the range of 20 to 90°C at distances greater than 40 mm from the beam's face. The temperature at Points 20 and 21 rose to a maximum of 90°C at 38 min in the Exposed connection while in the Concealed connection the temperatures recorded were higher by about 33% as they rose to more than 120°C at the same points

and same time of exposure due to heat conducted by the concealed plate to the core of the beam.

Figure 159 – The temperature distribution along beam width vs. time- Test 3.1

Similar to the Concealed connection, the recorded temperatures at the mid-span for the Exposed connections were lower than those near the connections, as shown in Figure 160. During the first 5 minutes, the temperature remained at ambient temperature, and then it started to increase. Points 29, 30, and 31 at the beam core did not record temperatures higher than 75°C, at failure. Also at Point 33, which is 20 mm from the top of the beam, recorded low temperatures due to the protection provided at the top of the beam. The temperature at the beam bottom corner, Point 25, rose to 100°C at 13 min and 300°C at 26 min and reached 750°C, at a failure time of 38.6 min.

Figure 160 – The wood's temperature vs. time at the mid-section - EX 4-12.7-60% - Test 3.1

The temperature profile along the depth of the beam at the mid-span is presented in Figure 161. The temperature of the beam's core remained at ambient temperature, except at the beam's bottom, 170 mm from the top, where the temperature increased above 500°C. Also it can be noticed that at 20 mm from the top, where insulation material was present, the temperature increased to 60°C. The temperature profile over the beam's width at the mid-span, presented in Figure 162, shows higher temperature at both sides of the beam. The maximum temperature at failure was 700°C, at the top corner (Point 35), and at the bottom corner of the beam (Point 25). The temperature at points deeper than 40 mm from the beam's surface ranged between 110°C to 210°C, at the time of failure, as shown in Figure 162.

Figure 161 – The temperature distribution along the beam's depth vs. time EX 4-12.7-60% - Test 3.1

Figure 162 – The temperature distribution along beam's width vs. time - Test 3.1

When the 19.1 mm bolt was used, similar temperature profiles for the wood beam near the connections and at the mid-span were observed, as shown in Figure 163 and Figure 164. In Test 4.2, which used the 19.1 mm bolts and a 100% load ratio, the maximum temperature at 20 mm was 350°C at the connections and 250°C at the mid-span. The core of the beam remained below 40 at the connections and did not exceed 35°C at mid span. It was also observed that the temperature recorded at Points 20 and 21 at the beam's core near the connections were 30% lower than those for the Concealed connections.

Figure 163 – The wood's temperature vs. time at the connections EX 4-19.1-100% - Test 4.2

Figure 164 – The wood's temperature vs. time at the mid-section - EX 4-19.1-100%- Test 4.2

5.2.4.2.1 Summary of the wood's temperature

Similar to the Concealed connections, the temperature was higher near the connections than at the mid-span due to the heat conducted by the plate and bolts. Temperatures at 20 mm from the surface were very high and ranged between 200°C to 750°C. The wood temperature at depths more than 40 mm from the surface ranged from 25°C to 100°C depending on the time of exposure. It was also observed that the temperature at the core of the beam near the connections recorded lower temperatures than the Concealed connections by about 30% to 33% due to less heat conduction to the core of the beam from the exposed plate.

5.2.5 Charring Rate

Table 7 and Figure 165, show the average charring rates at the mid-span and near to the connections. The depth charring rate at mid-span ranged from 0.61 to 0.78 with an average of 0.68 mm/min and the width charring rate ranged from 0.59 to 0.77 with an average of 0.70 mm/min. The average charring rate at the connections was 0.73 mm / min along the width. The rate along the depth of the connections was higher at 0.8 mm/min. It is observed that the charring rate is not different than those of the Concealed connections.

Table 7 -Table 3 - Charring rates for Group 3 and 4

Test #	Time min	Charing at mid-span				Charing at connection			
		Depth mm	Rate mm/min	Width mm	Rate mm/min	Depth mm	Rate mm/min	Width mm	Rate mm/min
3.1	38.60	23.5	0.61	22.83	0.59	*		23.5	0.61
3.2	22.15	17.33	0.78	15.5	0.70	*		18.75	0.85
4.1	26.00	18.33	0.71	19	0.73	21	0.8	19.83	0.76
4.2	18.00	11.33	0.63	13.83	0.77	*		12.5	0.69
Average			0.68		0.7				0.73

*In the test splitting of wood was observed at both connections therefore the depth measurement was not relevant

Figure 165 – The charring rates at the mid-span and the connection for Groups 3 & 4

5.2.6 Failure time and Failure Mode

The governed failure mode, in ambient temperature design, for the Exposed shear tab connection was a splitting failure for Group 3, which used four 19.1 mm bolts and yielding failure in Group 4, which used four 12.7 mm bolts.

In Test 3.1, the beam failed at 38.6 min, 43% more than the Concealed connection with the same load ratio and bolt diameter. Failure was characterized by a splitting of the wood at the connection at one side at the top bolt holes, with group tear out underneath the bottom bolts. Splitting at the top holes also started but did not propagate to the mid-span. Severe elongation and charring around the holes was observed and a large amount of bending was noticed on the bolts. (See Figure 166.)

Figure 166 - failure mode, Test 3.1 - a) wood splitting, b) bolt holes elongation, c) bolts bending

The specimen in Test 3.2 failed due to wood splitting at the connection at one side.

Splitting started at the bottom bolt holes and propagated to the mid-span, see Figure 167.

The failure time was 22.15 min, 21% more than the Concealed connections with the same

load ratio and bolt diameter. Severe elongation and some charring were noticed around the holes and moderate yielding of the bolts was also observed. (See Figure 168.)

Figure 167- The wood splitting propagated to mid-span - Test 3.2

a) Hole elongation and wood splitting at the bottom holes

b) Hole elongation and bending in the bolts

Figure 168 - a) wood splitting , b) bolts hole elongation and bolts bending- Test 3.2

For Test 4.2, the beam failed at 18.00 min, 38% more than the Concealed connections with same load ratio and bolt diameter. The failure was due to wood splitting, at one side of the bottom holes, which propagated to the mid-span. Minimal elongation and charring

around the bolt holes were noticed. (See Figure 169.) No bending of the bolts or plates was visible. (See Figure 170)

Figure 169 – The wood splitting at one side - Test 4.2

Figure 170 - No bending of the bolts - Test 4.2

5.2.6.1 Summary of the failure time and mode

The failure mode and comments are summarized in Table 8 for all the specimens in Groups 3 & 4. Similar to the Concealed connections, the predominant failure mode was splitting of wood parallel to grain at the connections. The cracks were initiated at the edges of the bolt holes, either at the top or bottom, due to the bolts acting as a wedge that

led to a concentration of lateral stresses at the edge of the bolt holes. This initiated the cracks as discussed in Section 5.1.6.1. Group tear out at the bottom bolts was also noticed. Pure brittle or pure ductile failure was not observed, it was a combination of elongation of bolts holes, bending of bolts and splitting of the wood in each case. It is believed that splitting of wood lead to the final failure. Similar to the Concealed connections, splitting propagated further to the mid-span at higher load ratios.

The charring around the bolts for the Exposed connections was less than for the Concealed connections with the same time of exposure. This is because the heat conducted into the beam's core was only due to the bolts conducting heat directly to the beam's core. The plate did not contribute directly to this thermal conduction.

Table 8 - Time and Failure mode for Group 3 & 4

Test #	# of Bolts	Bolt diameter	Load ratio %	Failure time-min	Failure mode
3.1	4	12.7	60	38.6	Splitting of the wood started at on side at the top bolts Group tear out of the bottom bolts at same side Elongation of the bolt holes Large bending of the bolts Charring around the holes
3.2	4	12.7	100	22.15	Splitting of the wood started at the bottom bolts at one side Elongation at the bolt holes Moderate bending of the bolts Charring around the holes
4.1	4	19.1	60	26.00	Splitting of the wood started at the bottom bolts at one side, and propagated to the mid-span Elongation of the bolt holes No bending of the bolts Charring around the holes
4.2	4	19.1	100	18.00	Splitting of wood started at the bottom bolts propagated to the mid-span, a decreased amount of splitting at the top bolts on the same side Minimum elongation of the bolt holes. No bending of the bolts Minor charring around the holes

It was noticed that the failure time increased with the decrease of the load ratio. Reducing the load ratio from 100% to 60% increased the failure time by about 44% and 74%, for the 12.7 mm bolts and 19.1 mm bolts. No bending of the 19.1 mm bolts was noticed. This was due to the high stiffness of these 19.1 mm bolts, while a large degree of bending was noticed for 12.7 mm bolts at the high exposure time.

5.2.7 Model validation

Tests were modeled using ABAQUS and the results were compared to the full-scale test results. Table 5 lists the models for Groups 3 and 4 for the Exposed shear connections with times of failure and failure modes.

Table 9 -Model test for Group 3 and 4

Group	Test	Connection Type	Bolt Dia mm	Bolt #	Load Ratio	Failure time min	Failure mode
3	3.1	Exposed	12.7	4	30%	51.1	A combination of the wood splitting and the bolts yielding
	3.2	Exposed	12.7	4	60%	39.2	A combination of the wood splitting and the bolts yielding
	3.3	Exposed	12.7	4	100%	18.8	A combination of the wood splitting and the bolts yielding
4	4.1*	Exposed	19.1	4	30%	40.7	The wood splitting
	4.2*	Exposed	19.1	4	60%	27.7	The wood splitting
	4.3*	Exposed	19.1	4	100%	14.8	The wood splitting
* These tests were carried out by Ohene (2014) [86] [9]							

5.2.7.1 Thermal Model

Test results of the full-scale test 3.1 were used to compare to the model predictions for Group 3. The temperature at Points T10 and T11 at the bolt shank were compared to the temperature predicted by the model as shown in Figure 171 (a & b.) The model predicted higher temperatures for the first 12 min, then closely aligned with the full-scale test results. However at T11, the model predicted a higher temperature at failure by about 10%. The higher temperature value predicted by the model could be due to the perfect contact assumed between the plate and the bolts which resulted in a faster conduction of heat to the bolt shank.

a) T10

b) T11

Figure 171 - Bolt temperature vs. time – a) T10 and b) T11 Group 3

The temperature in the wood section at Points 24, and 25 predicted by the model was compared to the full-scale test results in Figure 172. There is good agreement between the results of the test and the model.

a) T24

b) T25

Figure 172 -The wood's section temp vs. time - a) T24, b) T25, Group 3

Tests in Group 4 were modeled by Ohene (2014) [86] and found good agreement between the model and the test results, as shown in Figure 173 and Figure 174. However the model tended to overestimate the temperature at Point T16 after 10 minutes of fire exposure. T16, near the connection plate, was affected by the heat conducted from the plate to the wood. The model assumed perfect contact between the plate and the wood, which might have resulted in faster heat conduction into the wood than in the full-scale test, where minor gaps may have been present at the plate-wood interface.

Figure 173 - Test vs. model temperatures at Points T18 and T20 for Group 4 from Ohene (2014) [86]

Figure 174 - EX Test vs the model's temperature at Point T16 for Group 4 Ohene (2014) [86]

The temperature increase through the section due to fire exposure and the charring layer thickness were also compared. Figure 175 a is a typical temperature contour as predicted by the model at the mid-span and near the connection (300 mm from the edge of the

beam) The temperature along the width and the depth of the beam were in agreement with the full-scale test results as shown in Figure 175. (b and c)

a) The temperature distribution at the mid-span at 39 min

b) The temperature distribution at the beam's depth at the mid-span.

c) Temperature distribution at the beams width near connection

Figure 175 – The temperature distribution a) model at mid-span and connection b) tests at mid-span
c) test near connections

The temperature at 20 mm from both sides of the beam were within the range of 600° - 700°C and within 25 - 90°C at the beam's core after 38 minutes of exposure. (See Figure 175 b and c). A similar range was predicted by the model, in orange and blue contour colors.

The char layer predicted by the model (zone with temperatures over 280°C, light blue to red contour colors) was about 27 mm (see Figure 175 a) which was 15% more than the test char layer (23.5 mm).

5.2.7.2 Failure time

The mode of failure for Group 3 was a combination of wood cracking and bolts yielding, which was also predicted by the model. Similar to the Concealed connections,

the estimated failure time was recorded by following the time in which the crack was generated and propagated in the wood. (See Figure 176) The splitting of the wood was also marked by the sudden raise in the curve of energy dissipated by damage (ALLDMA) at 51 min as shown in Figure 177. The strain on the bolts exceeding the steel's yielding strain was also predicted by the model. (See Figure 178)

Cracks at bolts hole edge predicted by

Figure 176 – The crack generated at bolt's edge

Figure 177 – The progressive damage of the Exposed connections in Group 3

Figure 178 - Strains on the bolts exceed the steel's yielding strain at 51.1 min

For Group 4, the mode of failure predicted by the model was a splitting of the wood which was marked by the sudden rise in the curve of the energy dissipated by damage (ALLDMA) with the time as shown in Figure 197 and reported by Ohene [10], [86].

Figure 179 – The progressive damage of the EX connection for Group 4, from Ohene [86]

Table 6 lists the time of failure from the model and the test for the Exposed connections in Groups 3 and 4. This is also illustrated in Figure 180. The model predicted the time to failure with a variation of 1.5% to 17.7%

Table 10- The test and model times of failure for Groups 3 and 4

Group	Test	Connection Type	Load Ratio	Failure time min		Variation
				Model	Test	
3	3.1	Exposed	30%	51.10		
	3.2	Exposed	60%	39.20	38.60	1.55%
	3.3	Exposed	100%	18.80	22.15	15.12%
4	4.1	Exposed	30%	40.70		
	4.2	Exposed	60%	27.70	26.00	6.58%
	4.3	Exposed	100	14.80	18.00	17.7%

Figure 180 – The failure times of the Exposed connections for Group 3 and 4; Test vs. Model

5.2.8 Discussion

In this section, the effect of different parameters on the failure time of the Exposed connections will be discussed

5.2.8.1 Load ratio effect

The relation between the load ratios and failure times for Groups 3 and 4 are shown in Figure 181. The failure time for each test is shown as symbols (model times for Tests 3.1 and 4.1 were considered since no full scale test was conducted) while the line represents the possible relation for each group with the formula, also included. Similar to the Concealed connections, the specimen with a load ratio of 30% performed for a greater duration than those with load ratios of 60% or 100%. For Group 3, the failure time increased from 22.15 to 51.1 minutes (131%) when the load ratio was reduced from 100% to 30%. For Group 4, the increase of the failure time was about 22 minutes (126%) for the same reduction in the load ratio. Reducing the load ratio from 100% to 60% increased the failure time by 16.45 minutes (74%) for Group 3 and 8 minutes (44%) for Group 4. Similarly, the reduction of the load ratio from 60% to 30% improved the failure time by 32% for Group 3 and 57% for Group 4.

Figure 181 – The time to failure vs. the load ratio of the Concealed connection for Groups 3 and 4

The effect of the load ratio on the-mid span deflection is seen in

and Figure 183 for Groups 3 and 4. As expected, the initial deflection at the mid-span increased by increasing the load ratios. In both groups, the initial deflection ranged between 5 mm to 10 mm. Similar to the Concealed connections at a 100% load ratio, the increase of deflection prior to failure was more pronounced in comparison with the load ratios of 30% and 60%. The final deflection prior to failure was about 65 mm for a load ratio of 60% and ranged between 35 mm to 50 mm for a load ratio of 100%. It was observed that at a 60% load ratio, the beam suffered more deflection than a load ratio of 100% in both groups due to the high stresses that were generated when a high load ratio was applied. This led to sudden and fast failure (shorter time) which is associated with lower deflection at mid-span.

Figure 182 – The mid-span deflection vs. time for Group 3

Figure 183 – The mid-span deflection vs. time for Group 4

5.2.8.2 Bolts diameter effect

The effect of the bolt diameter on the failure time for the Exposed connections in Groups 3 and 4 was similar to those of the Concealed connections in Groups 1 and 2. In Groups 3 and 4, four bolts were used with a diameter of 12.7 mm and 19.1 mm respectively. Figure 184 shows the effect of the bolt diameter. Along with the load ratios, all other parameters remained the same for both groups. The figure shows that by reducing the bolt's diameter the failure time increased. The time improved by 10.4 minutes for a load ratio of 30%, 12.6 minutes for 60%, and 4.15 minutes for 100%. Increasing the bolt diameter from 12.7 mm to 19.1 mm reduced the failure time by 23% – 48%. It can be seen that the reduction in failure time was lower at the highest load ratio 100%, which is due to the shorter failure time at this load ratio.

The effect of the bolt's diameter on the mid-span's deflection for the Exposed connections in Group 3 and 4 was similar to the Concealed connections in Groups 1 and 2 as seen in Figure 185. At a load ratio of 60% there was no difference in the mid-span deflection in the first 20 minutes between the 12.7 mm and the 19.1 mm bolts. However, with continued exposure, the connection using the 19.1 mm bolts showed almost double the value of deflection at the same time of exposure. However the final deflection at failure was the same. (about 65 mm) It is believed that the higher stiffness of the larger diameter bolts resulted in a concentration of stresses on the wood fibers, which lead to faster failure in the wood, which is associated with higher values of deflection. On the contrary, when the load ratio increased to 100%, both curves for the 12.7 mm and 19.1 mm bolts were nearly parallel. The effect of the bigger bolts was not noticeable except in the initial deflection, which was lower by about 45%. Such a large difference was not expected. This could have been due to a small bending in the 12.7 mm bolts at the initial loading.

Figure 184 -The effect of bolt diameter on the failure time for Groups 3 and 4

Figure 185 – The mid-span deflection vs. time for Groups 3 and 4

5.3 Tests of the Seated Connections for Group 5

This section describes the results of the Seated connections, whose details were previously described in Section 3.4.3. The Exposed connection plate and the heat transfer to the wood were similar in both the Seated and the Exposed connections. The bearing plate provided extra protection to the beam at the connections during the initial time of exposure. Due to the similar results between the Seated and the Exposed connection tests, the furnace temperature, load ratio, specimen temperature profile and charring rate will not be discussed in this section.

5.3.1 Deflection

The mid-span deflection vs. time for Test 5.2, SE-1-12.7-100%, is plotted in Figure 186. The initial deflection at the mid-span with an applied load ratio of 100% was about 6.0 mm. This was less than the Exposed connection by 40%, and was due to the extra support the bearing plate provided to the beam at connection. The deflection of the beam after 15.9 minutes of exposure was 44 mm prior to failure. However, it increased to 58 mm at the failure time. The partial deflection recovery of both sides of the beam was observed during the first 2 minutes of exposure. Similar to the Exposed connections when the load ratio was reduced to +60% the initial deflection was decreased to 2 mm. The final deflection was nearly identical at 56 mm. (See Figure 187).

Figure 186 – The beam's mid-span deflection vs. time SE 1-12.7 100% - Test 5.2

Figure 187 – The beam's mid-span deflection vs. time SE 1-12.7 60% - Test 5.1

Test 5.3 was similar to test 5.1 except that the bolt was removed from the connection. The failure occurred at the steel plate rather than the wood beam. Figure 188 shows a relation between the mid-span deflection vs time for Test 5.3. A steep increase of the deflection after 15 minutes of exposure can be seen which is consisted of the beam deflection plus the plate deformation. The initial deflection at the mid-span increased to 4 mm about 100% more than the deflection of Test 5.1 (2mm). This was because there were no bolts to restrict the rotation of beam at the connection points. The final deflection at the mid-span does not represent the actual beam deflection because extra deflection was recorded due to the failure of the steel plate at the connection.

Figure 188 – The beam's mid-span deflection vs. time SE No Bolts- 60% - Test 5.3

5.3.2 Failure Time and Failure Mode

The configuration of the Seated connection significantly reduced the stress perpendicular to the grain because the major vertical loads applied were transferred

through the bearing plate to the column rather than through the bolts to the shear plate and column. This was the case with the Exposed and the Concealed connections. The governed failure mode, in ambient temperature design, for the Seated connections is flexural failure. As such the beam maximum capacity is higher than the Concealed and the Exposed connections. The load associated with 100% load ratio, is 80 kN. This is 11% higher than the load of the Concealed connections and 28% higher than the load of the Exposed connections.

The beam in Test 5.1, which had 60%, applied load ratio, failed at 34.18 minutes which was 26% higher than the failure time of the Concealed connections (with 4-12.7-mm bolts), and 31% higher than the failure time of the Exposed connection (with 4-19.1-mm bolts) at the same load ratio. However it was 12% less than the failure time of the Exposed connections with the 12.7 mm bolts. The wood started to crack at the bolt location followed by flexural / bending failure. The crack started at the beam's mid-span and propagated diagonally to the top of the beam as shown in Figure 189 (a and b.) Bending was noticed on the 12.7 mm bolts with elongation of the bolt holes as shown in Figure 189 c.

a) The beam in furnace after the test

b) The crack at the mid-span of the beam

c) Bolt bending and elongation in the bolt hole

Figure 189 – The beam failure Test 5.1 a, b, and c

Similar to Test 5.1, the beam in Test 5.2, which had 100% of the applied load ratio, failed by bending at 15.9 minutes. The short failure time was due to the high load ratio applied. However, the failure time was more by 22% than the Concealed connections with the same load ratio and using the 19.1 mm bolt. Two types of cracks were noticed on the beam. The first was splitting of the wood at the bolts depth followed by mid-span bending and cracking, which lead to failure. Bolt bending and bolt hole elongation were also noticed. (See Figure 190 a, b, and c)

a) The beam in the furnace after the test

b) The crack at the mid-span of the beam

c) No bending of the bolts or elongation in the bolt hole

Figure 190 –The beam failure Test 5.2 a, b, and c

Test 5.3 was similar to Test 5.1 except that the bolts were removed from the connection to reduce heat transfer to the beam core. This was also done to eliminate the load, which was applied perpendicular to the grain through the bolts that initiated the wood splitting. After 32 minutes of exposure, the beam did not show any sign of wood splitting or

bending cracks. However, failure occurred at the steel plate bracket where the welding failed at the connection between the bearing plate, side plate and the base plate as shown Figure 191.

a) The steel plate failed and the beam showed no signs of splitting or bending cracks

b) The welding between the bearing plate, the side plates and the base plate failed

Figure 191 – The steel plate failure Test 5.2 a, and b

5.3.2.1 Summary of time and failure mode

Table 11 summarizes the failure time and helps explain the failure mode for all tests in Group 5. When bolts were used at the connection, failure was caused by bending

at the mid-span which then caused the wood to split. When the beam did not have bolts at the connection, failure took place at the steel plate bracket.

When the load ratio was reduced from 100% to 60%, the failure time increased by 115%.

The high load ratio resulted in higher tension stress at the beam's bottom fibers. This coupled with fire exposure and charring, further reduced the thickness of the bottom layer, which led to failure.

The Seated connection outperformed the Concealed connection at the same load ratio by about 22 - 26%. The time of failure for the Seated connection did not vary greatly compared to the Exposed connection when tested using the 12.7 mm bolts. However, the Seated connection failure time was 31% greater than the Exposed connection failure time using the 19.1 mm bolts at the same load ratio.

Table 11 -Time and Failure mode for Group 5

Test #	# of Bolts	Bolt diameter	Load ratio %	Failure time min	Failure mode
5.1	1	12.7	60	34.18	Splitting of wood started at the bolts depth followed by bending cracks at the mid –span which propagated to the top of the beam Elongation of the bolt holes Bending of the bolts Charring around the holes
5.2	1	12.7	100	15.9	Splitting of wood started at the bolt depth followed by bending cracks at the mid –span propagated to the top of the beam Minimum elongation of the bolt holes. Minor bending of the bolts Minimal charring around the holes
5.3	No bolts	-	60	32	The steel plate failed at the welding No splitting or bending cracks on the wood beam

5.3.3 Model Validation

The Seated connections were modeled using ABAQUS . Table 12 lists the model tests for Group 5, which were performed by Ohene (2014) [86]. Group 6 includes the data for the model test without the bolts while Figure 192 shows the model assembly.

Table 12 - Model test for Groups 5 and 6

Group	Test	Connection Type	Bolts Dia mm	Bolts #	Load Ratio	Failure time min	Failure mode
5	5.1*	Seated	12.7	1	30%	53.7	Wood splitting
	5.2*	Seated	12.7	1	60%	35.5	Wood splitting
	5.3*	Seated	12.7	1	100%	16.5	Wood splitting
6	6.1	Seated	No	-	60%	57.2	Wood splitting
	6.2	Seated	No	-	100%	35.9	Wood splitting
* These numerical tests were carried out by Ohene (2014) [86] [9]

Figure 192- Seated Connection Assembly without the bolts

5.3.3.1 Thermal Model

The results of Test 5.3 were used to validate the thermal model for Group 6.

Figure 193 compares the increase in temperature at Points 25 and 19 as predicted by the model, to the increase in temperature during the full-scale test. There is good agreement between the results of the test and the model. The temperature distribution over the wood section and the predicted char layer were also in good agreement with the full-scale test results and did not differ from the Concealed and the Exposed connections hence it will not be discussed in this section.

a) T25

a) T19

Figure 193 – The wood's section temp vs. time - a) T25, b) T19, Group 6

5.3.3.2 Failure time

The perpendicular to grain stresses at the bolt and wood contact region in the Seated connection tests were reduced, because unlike the Exposed and the Concealed Connection, the applied load path from the beam to the column is through the base plate and not through the bolts. However, the bolts at the connection hold the beam sides from rising up because of the applied load at the 1/3 mid- section of the beam which generates the perpendicular to grain stresses at the bolt and wood contact region. These stresses were eliminated in the test with no bolts. Unlike the Exposed and the Concealed connections, wood splitting that caused the failure in the Seated connections was shifted from the bolt location and started at the applied load point, which was located at 1/3 of beam span. At this location both the shear and bending moment stresses are maximum which created the crack in the wooden beam. This was predicted by the model as shown

in Figure 194. However, the model underestimated the extent of the crack propagation and the inter-lamella split, that was observed during the test, because the model did not account for the influence of the glue between the lamellae of the glulam beam.

Figure 194 – The damage and failure at 1/3 of the beam span

The model predicted the yielding of the steel plate. (See Figure 195) However it did not predicted the welding failure because the seated bracket was modeled as a single piece rather than many pieces welded together.

Figure 195 - Yielding of the steel plate

The estimated failure time was marked by the sudden rise in the crack dissipated energy (ALLDMA) as shown in Figure 196. Table 13 indicates the time of failure for the model and the test, for the Seated connections, in Groups 5 and 6. This is also illustrated in Figure 197. The model predicted the time to failure with a maximum variation of 3.81%.

a) Seated connection with bolts (Ohene) [85]

b) Seated connection without bolts

Figure 196 – The progressive damage of the SE Connections a and b

Table 13 -Test and model time of failure for the Seated connections, Groups 5 and 6

Group	Test	Connection Type	Load Ratio	Failure time in minutes		Variation
				Model	Test	
5	5.1	SE with the bolts	30%	53.7	-	
	5.2	SE with the bolts	60%	35.5	34.18	3.86%
	5.3	SE with the bolts	100%	16.5	15.9	3.77%
6	6.1	SE without the bolts	60%	57.2	32.00	N/A*
	6.2	SE without the bolts	100%	35.9	-	-
*Test 6.1 The failure time represent the steel plate failure and not the wood beam failure

Figure 197 – The failure time of the Seated connections for Groups 5 and 6; Test vs. Model

5.3.4 Discussion

This section will discuss the effect of the load ratios and the presence of the bolts in the Seated connections.

5.3.4.1 Load ratio effect

The relation between the load ratios and tests failure times for Groups 5 and 6 are shown in Figure 198. The failure time for each test is shown using symbols while the lines represent the possible relationship between each group along with the corresponding formula. The formula for Group 6, can be used to extrapolate the failure time for a load ratio of 30%

$$t_f = 115.04 \text{ Exp } (-1.165\eta) \quad \text{Equation 11}$$

where:

t_f : the failure time in minutes

η : the load ratio

Using Equation 11, the failure time for the Seated connections without bolts and a load ratio of 30% is calculated to be 81.1 minutes.

The failure time of Group 5, (Seated connections with bolts), increased from 15.9 to 53.7 minutes (237%) when the load ratio was reduced from 100% to 30%. For Group 6 (Seated connections without bolts) the increase of failure time was 45.2 minutes (126%) for the same reduction in load ratio as described in Group 5. Reducing the load ratio from 100% to 60% increased the failure time by 19 minutes (115%) for Group 5 and 21.3 minutes (59%) for Group 6. Similarly, the reduction of the load ratio from 60% to 30% improved the failure time by 51% for Group 5 and 41% for Group 6.

Figure 198 – The failure time vs. the load ratio of the Seated connections for Groups 5 and 6

The effect of the load ratio on the mid-span deflection is shown in Figure 199. The initial deflection at the mid-span increased by 4 mm, when the load ratio was raised from 60% to 100%. Like the Concealed and the Exposed connections, the increase of deflection in the Seated connections was rapid and faster at higher load ratios. The final deflection, prior to failure, ranged between 55 mm and 60 mm.

Figure 199 – The mid-span deflection vs. time for Group 5

5.3.4.2 Bolt effect

The effect of the bolts on the failure times for Groups 5 and 6 is represented in Figure 200. All of the test parameters were similar for both groups excluding the bolt removal from the connections in Group 6. By removing the bolts, the failure time improved by 51% for a load ratio of 30%, 67% for a load ratio of 60%, and 125% for a load ratio of 100%. The presence of the bolts in the connection increased the heat conducted to the core of the beam, which caused charring and a reduction in the section's capacity. Moreover, removing the bolts eliminated the stress applied perpendicular to the wood grain at the wood bolt contact area as discussed previously in Section 5.3.3.2. The improvement of the failure time when the bolts were removed increased at the higher load ratios because the level of stress at the bolt and wood contact region increases with higher applied load and it leads to faster failure.

Figure 200 - The effect removing the bolts on the failure times for Groups 5 and 6

5.4 Tests of the Exposed and the Concealed Connections with Two Bolts - Group 6

This section describes and discusses the results of the Exposed and the Concealed connections in Group 6. The tests studied the affect of the number of bolts on the failure time for the Concealed and the Exposed connections. Two full-scale tests were carried out, one for each type of connection. Two 12.7 mm bolts were used with a load ratio of 100%. The validated model was used to predict the failure time of these connections at load ratios of 30%, 60%, and 100%. 12.7 mm and 19.1 mm bolts were used for the model. Figure 201 (a & b), shows a typical specimen in the furnace before and after the test.

a) Before the test in the furnace

b) After the test in the furnace

Figure 201 – The Exposed connections with the two bolts - Group 6

The furnace temperature, load ratio, specimen temperature profile and charring rate results are similar to the Exposed and the Concealed connections with four bolts which were discussed previously in Sections 5.1 and 5.2. Therefore they are not discussed in this section. Deflection, failure time, failure mode and model results will be discussed in this section.

5.4.1 Deflection

The mid-span deflection for Tests 6.1 and 6.2 are shown in Figure 202 and Figure 203, respectively. The initial deflection at the mid-span was 6.5 mm for the Exposed connection and 8 mm for the Concealed connection. However, the final deflection for the Exposed connection was 65 mm which was greater by 10 mm compared to the Concealed

connections. This difference was due to an additional 2.5 minutes of exposure during the Exposed connections test. The failure time was 20 minutes for the Concealed connections and 22.5 minutes for the Exposed connections. In both instances, the applied load ratio was 100%.

Figure 202 – Mid-span deflection of the beam vs. time CN 2-12.7 100% - Test 6.1

Figure 203 – Mid-span deflection of the beam vs. time EX 2-12.7 100% - Test 6.2

5.4.2 Failure Time and Failure Mode

Reducing the number of bolts did not have any effect on the failure time for the Exposed connections which was 22.5 minutes for the connections with 2 bolts and 22.15 minutes with 4 bolts. It had a minor effect on the Concealed connections that had failure times of 18.25 minutes for 4 bolts and 20 minutes with 2 bolts.

The increase in failure time was even less significant for higher load ratios when the applied load reaches the section's ultimate capacity at ambient conditions. This was because the failure time is usually short and fire exposure is at a minimum. Therefore the high level of mechanical stress applied to the section was the principle cause of failure, rather than fire exposure.

The failure modes for the Concealed and the Exposed connections were both attributed to the yielding of the bolts and the splitting of the wood at the bolt holes which propagated to the mid-span as shown in Figure 204 and Table 14. The failure mode, which occurred during the tests with 2 bolts, was similar to the one in the tests with 4 bolts in both the Concealed and the Exposed connections; however, bolt yielding was more significant when only 2 bolts were used. This occurred because the vertical load that was transferred from the beam to the plate through a fewer number of bolts lead to a greater amount of stress on each bolt.

5.4.3 Model Validation

ABAQUS was used to model the Concealed and the Exposed connections with 2 bolts. (See Figure 205 a & b) The validated model assembly was used to predict the failure time for the Exposed and the Concealed connections that used 2 bolts, one pair of

12.7 mm and one pair of 19.1 mm. The load ratios of 30%, 60% and 100% were tested as shown in Table 15.

Figure 204 – The typical failure mode for the Exposed connections with 2 bolts

Table 14 - Time and Failure mode for Group 6

Test #	# of Bolts	Bolt diameter	Connection	Load ratio %	Failure time min	Failure mode
6.1	2	12.7	Concealed	100%	20.0	Splitting of wood started at the bolt hole locations propagated to the mid-span Minor elongation of the bolt holes Bending of the bolts Charring around the holes
6.2	2	12.7	Exposed	100%	22.5

Figure 205 - 2 bolt connection model assembly

Table 15 - Test and model time of failure for the 2 bolt connections in Groups 7, 8, 9 and 10

Group	Test	Connection Type	Bolts Dia mm	Bolts #	Load Ratio	Failure time min	Failure mode
7	7.1	Concealed	12.7	2	30%	45.5	Wood splitting and bolt yielding
	7.2	Concealed	12.7	2	60%	37.4	
	7.3	Concealed	12.7	2	100%	23.0	
8	8.1	Exposed	12.7	2	30%	48.5	
	8.2	Exposed	12.7	2	60%	40.0	
	8.3	Exposed	12.7	2	100%	24.4	
9	9.1	Concealed	19.1	2	30%	40.0	
	9.2	Concealed	19.1	2	60%	35.4	
	9.3	Concealed	19.1	2	100%	22.5	
10	10.1	Exposed	19.1	2	30%	44.10	
	10.2	Exposed	19.1	2	60%	36.8	
	10.3	Exposed	19.1	2	100%	23.5	

5.4.3.1 Failure time

The amount of heat conducted to the core of the wooden beam was reduced when 2 bolts were used at the connection rather than 4 bolts. The number of drilled holes was 50% less which led to an improved failure time for the connection with 2 bolts. The failure mode was a combination of wood splitting and yielding of bolts. The wood

splitting was predicted by the model, as shown in Figure 206. The failure time predicted by the model is evidenced by the sudden rise of energy dissipated by damage (ALLDMA) as shown in Figure 207. Bolt yielding was also predicted by the model, as shown in Figure 208. Table 16 compares the failure times predicted by the model with the failure times recorded during the tests. The model predicted the failure time for the connections with 2 bolts within a range between 8.5-13%.

Figure 206 –Wood splitting as predicted by the model for the 2 bolt connections

Figure 207 –The typical progressive damage curves of the CN Connection with 2 bolts

Figure 208 – The bolts yielding as predicted by the model for the 2 bolt connections

Table 16 - Test and model time of failure for 2 bolts connections for Group 6

Group	Test	Connection Type	Load Ratio	Failure time min		Variation
				Model	Test	
6	6.1	Concealed with the two 12.7 mm bolts	100%	23.0	20.0	13%
	6.2	Exposed with the two 12.7 mm bolts	100	24.4	22.5	8.5%

5.4.4 Discussion

This section discusses the effect of the load ratios and the bolts' sizes on the performance of the Exposed and the Concealed connections with 2 bolts.

5.4.4.1 Load ratio effect

Figure 209 and Figure 210 show the relationship between the load ratios and failure time for the Concealed connections, which used two 12.7 mm bolts and the Exposed connections, which used two 19.1 mm bolts. The failure time for each test is shown using symbols (model times were considered where no full scale test was conducted) and the dotted line represents the possible relation for each group. The failure time improved by 128% when the load ratio was reduced from 100% to 30% for the Concealed connections using 12.7 mm bolts and by 63% for connections with the 19.1 mm bolts. Reducing the load ratio from 100% to 60% increased the failure time by 87% for the test using the 12.7 mm bolts and by 44% for test using the 19.1 mm bolts.

Similarly, for the Exposed connections test, when the load ratio was reduced from 100% to 30%, the failure time increased by 116% using 12.7 mm bolts and by 88% for the test using the 19.1 mm bolts. When the load ratio was decreased from 100% to 60%, the failure time increased by 78% when the 12.7 mm were used and 58% when the 19.1 mm bolts were used.

Figure 209 – Failure time vs. the load ratio of the CN connections using two bolts

Figure 210 – The failure time vs. the load ratio of the EX connections with two bolts

5.4.4.2 Bolts Diameter Effect

The effect of the bolt diameter on the failure time for the Concealed connections is shown in Figure 211 while Figure 212 shows the same effect for the Exposed connections. Increasing the bolt diameter from 12.7 mm to 19.1 mm had a similar effect for both types of connections. For the Concealed connections, the failure time was slightly reduced by 2% when the load ratio was 30%, by 6% when the load ratio was 60%, and by 14% when the load ratio was 100%. For the Exposed connections the failure time was slightly reduced by 4% when the load ratio was 30%, by 9% when the load ratio was 60%, and by 10% when the load ratio was 100%.

Similar to the connection tests using 4 bolts, the reduction in failure time was the lowest when the load ratio was 100%. However, unlike the connections using 4 bolts, increasing the bolt diameter did not reduce the failure time significantly. Using a fewer number of bolts reduced the amount of heat conducted into the wooden beam's core and hence the degradation of the entire wood section increasing the failure time.

Figure 211 - The effect the bolt diameter had on the failure time for the CN connections using 2 bolts

Figure 212 - The effect of bolt diameter had on the failure time for the EX connections using 2 bolts

5.5 Comparison of Failure Time of the Different Connections

In this section, the failure times of the Concealed connections (CN) the Exposed connections (EX) and the Seated connections (SE) will be compared in order to evaluate their performance under various applied load ratios as well as the number of bolts used.

5.5.1 Effect of the Number of Bolts

The presence of the bolts in the connection affected the connection's capacity and also played a significant role in transferring heat to the core of the wooden beam. Figure 213 (a) and (b) represent the effect of the 2 or 4 bolts on the failure time of the CN connection tests using 12.7 mm and 19.1 mm bolts. When the number of bolts were decreased from 4 to 2, the failure time increased by a range of 19% to 86% for the 19.1 mm bolts and a range of 10% to 38% for the 12.7 mm bolts. The improvement in failure time was more noticeable for the connections with larger bolts because they required larger holes in the wood section. They also conducted more heat to the wooden core of the beam and had a larger contact area, where the perpendicular to grain stress was generated.

All of the above factors expedited the degradation of the wood section under fire exposure. Hence, by reducing the number of bolts by 50% enhanced the failure time during the tests of the CN connections.

a) The CN connections using the 19.1 mm bolts

b) The CN connections using the 12.7 mm bolts

Figure 213 – The effect of the number of bolts on the failure time of the CN connections

The same trend was also observed for the EX connections using the 12.7 mm and 19.1 mm bolts as shown in Figure 214 (a) and (b). Reducing the number of bolts from 4 to 2, increased the failure time by a range of 8% to 40% in the tests using the 19.1 mm bolts and from 1.5% to 4 % in the tests using the 12.7 mm bolts. There was a smaller improvement of the failure time for the EX connections compared to the CN connections. The number of bolts had a less significant effect when the thickness of the wood was greater, as with the EX connections due to the absence of a notch in the wooden beam that accommodated the plate in the CN connections

a) The EX connections using the 19.1 mm bolts

b) The EX connections using the 12.7 mm bolts

Figure 214 – The effect of the number of bolts on the failure time of the EX connections

5.5.2 Failure Times for the Different Load Ratios

Figure 215 shows the failure times for the load ratios used for the CN and the EX connections which had four 19.1 mm bolts, as well as the SE connections which used one 12.7 bolt and the SE connection without a bolt. In Figure 215 (a) the failure time for each test is shown using symbols, while the line represents the possible numerical relationships for each group. Figure 215 (b) shows a bar chart of the failure time. The SE connections without a bolt had the highest failure time compared to the SE connection with bolts (second highest), the EX connection (third highest), and the CN connection, which had the lowest failure time. Comparing the failure time of the CN connection to the failure time of the SE connection without a bolt it can be observed that the SE connection had a failure time that was 142% higher for the 30% load ratio, 200% for the

60% load ratio, and 176% for the 100%. The EX connection also performed better than the CN connection. The failure time for the EX connection was 21% higher for the 30% load ratio, 36% for the 60% load ratio, and 38% for the 100% load ratio.

When the applied load reaches the ambient design capacity of the connections at the load ratio of 100%, there were no significant differences between the failure times for the CN, the EX and the SE with a bolt connections, as shown in Figure 215. The failure time did not improve greatly due to the high level of stress present, at bolt and wood contact area which dominate the connection failure as opposed to degradation of the connection due to the exposure to fire. This conclusion can be drawn since all of the connections failed in a relatively short time and there was no substantial charring of the wood. The improvement of the failure time comparing the CN, the EX and the SE without a bolt connections were more noticeable when the load ratios applied were 30% or 60%.

The configuration of the connections had a significant effect on the failure time of the connections. For the CN connections, the center notch used to accommodate the steel plate reduced the thickness of the wood section and hence its capacity.

a) Failure time curves of the SE, EX and CN connection using 4-19.1-mm bolts and corresponding formula

b) Failure times of all connections

Figure 215 – Failure times for the SE, EX, and CN connections using 4- 19.1- mm bolts

In addition, the heat was conducted more quickly through the plate to the core of the wooden beam, which in turn, shortened the failure time. The external steel plate used in the EX connections eliminated the need for the center notch. The steel plate provided partial protection to the wood during fire exposure. This reduced the level of charring which increased the failure time.

The SE configuration was similar to the Exposed connection and the bearing plate underneath the beam provided additional protection from heat exposure to the bottom of the beam. In addition, the use of only one bolt reduced the heat transferred to the core of the wooden beam and results in a higher failure time of the assembly. When the bolt was removed from the SE connections, there was significant improvement in the failure time. This was 22.9 minute greater than the CN Connections at a load ratio of 100%, because no heat was transferred to beam's core and the perpendicular to the grain stress generated at the bolt-wood contact was eliminated.

The same trend of failure time improvement was observed for the CN, the EX with four 12.7 mm bolts and the SE connections with and without a bolt. (See Figure 216 a & b) The SE connection without the bolts outperformed the CN and EX connections for all of the different load ratios.

The failure times for the SE connections with the bolt, were greater than the CN connection by 101% for a load ratio of 30%, by 111% for a load ratio of 60%, and by 96% for a load ratio of 100%. The EX connections had better failure times than the CN connections by 26% for a load ratio of 30%, by 42% for a load ratio of 60%, and by 21% for a load ratio of 100%.

a) Failure time curves for the SE, CN and EX connections with 4-12.7-mm bolts and corresponding formula

b) The relevant failure times

Figure 216 – Failure times for the SE, EX and CN connections with 4-12.7-mm bolts

Unlike the EX connections using the 19.1 mm bolts, the EX connections using the 12.7 mm bolts had a greater failure time compared to the SE connections with the bolts. The time was better by 4.42 minutes for a load ratio of 60% and by 6.25 minutes for a load ratio of 100%. Using smaller diameter bolts improved the failure time of the EX connections; however, the SE connections without bolts still outperformed the EX connections with 12.7 mm bolts. The failure time of the SE connection was 58% higher for a load ratio of 30%, 48% for a load ratio of 60%, and 62% for load ratio of 100% than the failure time of the EX connections.

To further investigate the failure times of these connections, the validated ABAQUS models were used to predict the failure times of the CN, EX and SE connections without a bolt, under the same applied loads. A load ratio of 100% for the EX connection (62.5 kN) was applied to all the connections and their respective failure times are tabulated in Table 17. The predicted failure times of the connections had the same order (CN, EX, and SE without bolts) seen in the tests. The SE connections without bolts outperformed both the CN and the EX connections by 133% and 126% respectively.

Table 17 – Predicted failure times for the EX, the CN and the SE connections with a 62.5 kN load

Test	Connection Type	Load kN	Failure time min
3.2	EX with the four 12.7 mm bolts	62.5	22.15
11.1	CN with the two 12.7 mm bolts	62.5	21.50
11.2	SE without bolts	62.5	50.2

5.5.3 Summary

Figure 217 depicts the failure times for the CN, the EX and the SE connections with different bolts sizes, bolt numbers, and load ratios.

Figure 217 – Failure times for the different types of connections

The connections using 12.7 mm bolts had a higher failure time than the connections using 19.1 mm bolts, while the connections with 2 bolts had a higher failure time than the connections with 4 bolts for the same load ratios. The EX connections had a higher failure time than the CN connections while the SE without bolts had a higher failure time than the EX connections at the same load ratios. For all applied loads, the SE without bolts connection had the largest failure time and the CN connection the lowest.

5.6 Empirical Relationship

Full-scale tests are the most accurate method to determine the failure time of structural elements, though they are very expensive. A less expensive alternative is the use of finite element modeling. However, configuring the parameters accurately is a complex process. Therefore, for design purposes, a simplified calculation method that involves variables, which directly affect the failure time of hybrid timber connections, is preferred. Peng [78] used this approach to estimate the failure time of timber connections under pure tension.

The tests and the finite element modeling results were used to develop formulas to estimate the failure times of the tested connections. The test results show that the failure time is a function of the load ratios, the size of the bolts, the number of the bolts and the wood member's thickness (the wood's thickness at both sides of the plate in the CN connections and beam's width in the EX and SE connections). The beam effective depth also affects the beam capacity and failure time, hence the function can be represented as:

$$t_f = f(R_f, t_1, d_b, d_e, n, \beta) \quad \text{Equation 12}$$

The following non- dimensional factors will be used:

$$t^* = \frac{t_f}{(\frac{t_1}{\beta})}, \quad \eta = \frac{R_f}{R_0}, \quad d^* = \frac{d_e}{n d_b}, \quad t_1^* = \frac{t_1}{d_b}$$

where:

t_f is failure time in minutes

t_1 is the width of the wooden member in mm

β is the charring rate in mm/min

R_f is the applied load in kN

R_0 is the design capacity in kN

d_e is the beam's effective depth in mm

d_b is the bolt's diameter in mm

n is the number of bolts

The following function proposed by Peng [78] will be used as a guide to develop simplified empirical relation for the CN, the EX and the SE connections

$$t^* = 1 - (\eta)^M (d^*)^N \quad \text{Equation 13}$$

Equation 13 is modified to include the non-dimensional factors proposed in Equation 12.

The proposed power function used in this study is:

$$t^* = 1 - (\eta)^M (t_1^*)^N (d^*)^K \quad \text{Equation 14}$$

5.6.1 Tests Data

In this section, the data used to develop the empirical functions for the three different types of connections will be discussed.

Table 18 lists the characteristics of the full scale and modeling tests such as failure time, bolt diameters, charring rate, the member's thickness and the effective depth used to develop the empirical functions for the CN, the EX and the SE connections. The average charring rate from the full-scale tests for each connection was used. For the CN connections, the wood member thickness used is the thickness of the wood at each side of the plate. For the EX and the SE connections the full beam's width was used as the wood member thickness. The effective depth used was the depth from top of the beam to the bottom bolts for the CN and the EX connections and the entire beam's depth for the SE connections.

The following assumptions were made when developing the empirical functions:

1. The charring rate is constant
2. The structural elements are exposed to standard fires
3. If no load was applied, the failure time was assumed to be the time required

for the sections to be completely charred $t_f = \frac{t_1}{\beta}$

Table 18 - Test data used to develop the empirical functions

Connection Type	Load Ratio η	Charring rate mm/min β	Side member thickness t_1 mm	Effective depth d_e mm	bolt diameter d_b mm	Number of bolts n
CN	0.3-1.0	0.76	64	146	12.7 & 19.1	2 & 4
EX	0.3-1.0	0.73	140	146	12.7 & 19.1	2 & 4
SE	0.3-1.0	0.73	140	191	12.7	1 & No bolt

5.6.2 Correlations for the CN and the EX Connections

The proposed formula of the failure time for the CN and the EX connections is

Equation 14:

$$t^* = 1 - (\eta)^M (t_1^*)^N (d^*)^K \quad \text{Equation 14}$$

The tests data was used to determine the individual values of M, N and K, for the CN and the EX connections using best-fitting by the method of least squares.

The connection factors computed are $M = 0.317$, $N = -0.007$ and $K = -0.231$ and the coefficient of determination $R^2 = 0.98$. The value of the coefficient of determination value is an indication of the relationship strength, $0 \leq R^2 \leq 1$. The higher the value indicates the stronger the relationship is. The correlation result is shown in Figure 218 for CN

connections. Equation 14 can be rearranged in a dimensional format to yield Equation 15 to calculate the failure time of CN connections:

$$t_f = \left[1 - \eta^{0.317} \left(\frac{t_1}{d_b} \right)^{-0.007} \left(\frac{d_e}{n d_b} \right)^{-0.231} \right] \frac{t_1}{\beta} \quad \text{Equation 15}$$

Figure 219 shows the relationship between the failure time results from the tests or the models and the time predicted by Equation 15. The correlations predicted the failure time within the boundary of $\pm 15\%$.

Figure 218 - Curve fitting analysis results for the CN connections

Figure 219 – The tested or modeled time vs. the formula predicted time for the CN connections

The correlated result for the EX connections is shown in Figure 220. The connection factors are $M = 0.115$, $N = -0.048$ and $K = -0.02$, and the coefficient of determination $R^2 = 1.00$, which indicates a strong relationship similar to the CN connections. Equation 15 can be rewritten using the EX connection factors to yield an equation to calculate the failure time of the EX connections.

$$t_f = \left[1 - \eta^{0.115} \left(\frac{t_1}{d_b} \right)^{-0.048} \left(\frac{d_e}{n d_b} \right)^{-0.02} \right] \frac{t_1}{\beta} \quad \text{Equation 16}$$

Figure 221 shows the relationship between the tested or modeled results and the time predicted by the correlation. Similar to the CN connections, the correlations predicted the failure time within a boundary of $\pm 15\%$. Table 19 summarizes the connection factor values for the CN and EX connections with the proposed correlation.

Table 19 – Correlation and connection factors of the CN and EX connections

$t_f = \left[1 - \eta^M \left(\frac{t_1}{d_b} \right)^N \left(\frac{d_e}{n d_b} \right)^K \right] \frac{t_1}{\beta}$			
Connection	M	N	K
CN	0.317	-0.007	-0.231
EX	0.115	-0.048	-0.02

Figure 220 - Curve fitting analysis results for the EX connections

Figure 221- The tested or modeled time vs. the formula predicted time for the EX connections

5.6.3 Correlations for SE Connections

The number of bolts was not a factor in the failure time for the SE connections with bolts since only one bolt was used. Therefore, the proposed non-dimensional formula (Equation 14) was modified to remove (n) to yield Equation 17.

$$t_f = (1 - \eta^M (d_e / d_b)^N) (t_1 / \beta) \quad \text{Equation 17}$$

For the SE connections without bolts, the diameter of the bolts (d_b) was also removed and replaced with the member's thickness (t_1) to yield Equation 18.

$$t_f = (1 - \eta^M (d_e / t_1)^N) (t_1 / \beta) \quad \text{Equation 18}$$

The correlation results are shown in Figure 222 and Figure 223 for the SE connections with and without bolts respectively. For the SE connections with bolts, the connection factors were $M=0.201$, $N= - 0.033$ and the coefficient of determination was $R^2 = 1.00$.

Figure 222 - Curve fitting analysis results for the SE connections with the bolts

Figure 223 -Curve fitting analysis results for the SE connections without the bolts

For the SE connections without bolts, the factors were $M=0.285$, $N= -0.669$ and the coefficient of determination was $R^2=1.00$. Therefore, Equation 17 and Equation 18 can be rewritten using the connections factors to yield;

$t_f = (1 - \eta^{0.201} (d_e / d_b)^{-0.033}) (t_1 / \beta)$ for the SE connections with bolts, and

$t_f = (1 - \eta^{0.285} (d_e / t_1)^{-0.699}) (t_1 / \beta)$ for the SE connections without bolts.

Figure 224 and Figure 225 show the relationship between the failure time results from the test or model and the time predicted by Equation 17 and Equation 18. The correlations predicted the failure time accurately.

Figure 224 – The tested or modeled time vs. formula predicted time for the SE connections with bolts

Figure 225 - Tested or modeled time vs. formula predicted time for the SE connections without bolts

An alternative correlation is also proposed to calculate the failure time for the SE connections with or without bolt using one equation as shown below:

$$t_f = [K \cdot \eta^M \{d_e / (t_1 / d_b)\}^N] (t_1 / \beta) \quad \text{Equation 19}$$

Six test results were used to determine the connection factors K, M and N for Equation 19, unlike Equation 17 and Equation 18 where only three test results were used.

Figure 226 shows the correlation results for the SE connections with and without bolts.

The connection factors were $K = 1.555$, $M = 0.136$, $N = 0.971$ and the coefficient of determination was $R^2 = 0.993$. The relationship between the test results and failure time predicted by Equation 19 is shown in Figure 227 the alternative correlation predicted the time with $\pm 10\%$.

Figure 226 -Curve fitting analysis results for the SE connections with and without bolts

Figure 227 - Tested or modeled time vs. formula predicted time for SE connections with and without bolts

Table 20 lists the proposed correlations with the associated connection factors M and N for the SE connections.

Table 20 - Correlation and connection factors M & N for the SE connections

SE connections with bolts	
$t_f = (1 - \eta^M (d_e/d_b)^N) (t_1/\beta)$	
M	N
0.201	-0.033
SE connections without bolts	
$t_f = (1 - \eta^M (d_e/t_1)^N) (t_1/\beta)$	
M	N
0.285	-0.669
SE connections with and without bolts	
$t_f = [1.555 - \eta^M \{d_e/(t_1 - d_b)\}^N] (t_1/\beta)$	
M	N
0.136	0.971

5.6.4 Correlation limitation

The proposed correlations can be used for fire design purpose of hybrid timber connections that satisfy the following.

1. Concealed , Exposed and Seated connections
2. Untreated glulam beams.
3. Not curved beams.
4. Standard time-temperature curve as per CAN/ULC –S101.
5. Expected failure: brittle/splitting of wood.

5.6.5 Summary

Five numerical formulas were developed to calculate the failure time of different connections; the formulas were able to calculate the time to failure within a range between 0 to ± 15 % as shown in Table 21. For design purpose and to ensure the time calculated by the proposed formula is always equal or less than the full-scale or model test results the design time is to be divided by a factor ranging from 1 to 1.15 as listed in the table.

Table 21 - Empirical formulas

Connection	Formulas	Design time
Concealed	$t_f = \left[1 - \eta^{0.317} \left(\frac{t_1}{d_b} \right)^{-0.007} \left(\frac{d_e}{n d_b} \right)^{-0.231} \right] \frac{t_1}{\beta}$	$\frac{t_f}{1.15}$
Exposed	$t_f = \left[1 - \eta^{0.115} \left(\frac{t_1}{d_b} \right)^{-0.048} \left(\frac{d_e}{n d_b} \right)^{-0.02} \right] \frac{t_1}{\beta}$	$\frac{t_f}{1.15}$
Seated with bolt	$t_f = \left[1 - \eta^{0.201} \left(\frac{d_e}{d_b} \right)^{-0.033} \right] \frac{t_1}{\beta}$	t_f
Seated without bolt	$t_f = \left[1 - \eta^{0.285} \left(\frac{d_e}{t_1} \right)^{-0.669} \right] \frac{t_1}{\beta}$	t_f
Seated with or without bolt	$t_f = \left[1.555 - \eta^{0.136} \left(\frac{d_e}{t_1 - d_b} \right)^{0.971} \right] \frac{t_1}{\beta}$	$\frac{t_f}{1.10}$

6 Summary, Conclusions and Recommendation

6.1 Summary

The objective of this research was to study the effects of different parameters on the fire performance of hybrid steel-timber connections and to develop simple numerical relations to calculate the failure time of the connections for design purposes.

To achieve the project objectives, fourteen (14) full scale fire tests were conducted and thirty one (31) finite element models were developed, using a three-dimensional numerical heat transfer model and an analytical structural model in ABAQUS /Standard V6.112, to simulate the fire resistance tests of the connections. The model couples heat and structural effects on the assembly by adopting the sequentially-coupled thermal-stress procedure in ABAQUS.

Three types of hybrid connections were investigated; the Exposed shear tab (EX), the Concealed shear tab (CN) and the Seated (SE), between a glulam beam and two pin-pin steel columns under standard time-temperature fire exposure in compliance with CAN/ULC-S101. The glulam beam spanned 1.9 m and was subjected to two-point loads at the mid 1/3 span. The beam was protected from the top and the steel columns were completely protected.

The parameters that were investigated were the connection type, size and number of bolts (2 and 4 bolts of 12.7 and 19.1 mm in diameter) and the load ratio (10%, 30% and 100%). The load ratio is the ratio between the applied load and the assembly ambient temperature design capacity.

The data recorded for all full scale tests included the temperature profiles of wood and steel sections, the deflection of the beam, the charring rate and the failure time. These data were used to validate the numerical heat transfer model and structural model. In general, the model predicted results were found to be in good agreement with the full scale test results for the CN, the EX and the SE connections.

The failure time is the primary concern and was determined during the tests as the time when the specimen could not withstand the applied load or the load dropped with no recovery in the specimen. The finite element model was able to track the progressive damage in the wood beam in the form of energy dissipated (ALLDMD) which represents the energy released in the formation of cracks and their propagation. The failure time was determined in the model as the time when a sudden rise in the energy from a low value to the peak value is observed in the damage dissipated energy curve.

The characteristics of the tests, the failure times from full the scale tests and the model predicted failure times were used to develop simple numerical formulas to predict the failure time for CN, EX and SE connections for design purposes. The formulas were able to calculate the times of failure within a range of ± 0 to 15%.

The test results showed that the failure time of the SE connection without a bolt was higher than the failure time of the EX and CN connections. Increasing the applied load to the ambient design capacity at 100% load ratio reduced the failure time to less than 40 minutes for all connections. Lowering the load ratio from 100% to 60% and then to 30%, reducing the bolts sizes and reducing the number of bolts from 4 to 2 increased the failure times. The final failure mode was always brittle / splitting of wood.

6.2 Conclusions

Based on the research results, the following conclusions can be drawn on the fire performance of hybrid timber connections:

- 1- The configuration of the connections has a significant effect on the failure time of the connections. For the CN connection, placing the steel plate in the center notch of the beam, reduced the wood member thickness and transmitted more heat to the core of the wood at the connection region which shortened the failure time. In the EX connection, the steel plates are placed at both sides of the beam which provide initial protection to the wood at the connection region, hence it enhanced the failure time. The base plate for the SE connection provided additional initial protection at the bottom of the beam.
- 2- The applied load path in the CN and the EX connections was through the beam, the bolts and the steel plate to the columns, which generated perpendicular to grain stresses at the wood-bolt contact area and led to faster failure. The applied load path in the SE connection was through the beam and the steel base plate to the columns. This reduced the perpendicular to grain stresses at the wood-bolt contact area that led to longer failure time. Further improvement in failure time was achieved by removing the bolts from the SE connection. The impact of removing the bolt from the Seated connection on the beam ambient design capacity such as in seismic design should be checked.

- 3- The test results show that the SE connection without a bolt had the highest failure times compared to the SE with bolt, the EX and the CN connections. The CN connection had the lowest failure time.
- 4- The test results show that the failure time of the SE connection without a bolt was higher than the failure time of the CN connection by a range of 101-142% for the 30% load ratio, 111-200% for the 60% load ratio, and 97-176% for the 100% load ratio. The failure time of the EX connection was higher than the CN connection by a range of 21-26 % for the 30% load ratio, 36-43% for the 60% load ratio and 21-38% for the 100% load ratio.
- 5- The test results show that increasing the load ratio reduced the failure time. The failure time reduced by a range of 12-45% when the load ratio increased from 30% to 60% and 43-69 % when the load ratio increased from 30% to 100%.
- 6- The failure time of the EX, the CN and the SE connections at 100% load ratio were less than 45 minutes which is a target rating for Canadian code compliance.
- 7- When the applied load reached the ambient design capacity, at the load ratio of 100%, the mechanical stresses dominated the failure time rather than the wood properties degradation due to fire exposure.
- 8- The test results show that using smaller diameter bolts increased the EX and the CN connections failure time for all the applied load ratios.
- 9- The test results show that the EX and the CN connections using 2 bolts had higher failure time compared to those using 4 bolts. The results also show that the SE

connection without a bolt had higher failure time than the SE connection with a bolt.

10- Partial deflection recovery at both sides of the beam was observed during the first 2–5 minutes of fire exposure. The beam deflection increased with time due to fire exposure until failure. .

11- The charring rate for the glulam beam measured during the tests was in a range of 0.63 to 0.76 mm/min. Charing was also observed at the center notch in the CN connection and around the bolts, when bolts were used.

12- The final failure mode was always brittle / splitting of wood even when ductile failure (yielding of bolts or crushing of wood) was the governing failure mode in the connection's ambient temperature design. The wood split initiated at the bolt hole and propagated to the mid-span. Elongation of bolt's holes and yielding of steel bolts were also observed on the tested beams.

13- The thermo-mechanical response of the hybrid timber-steel connection under fire exposure can be predicted using a sequentially-coupled thermal-stress analysis procedure in a finite element model. The model was validated using full scale test results.

14- To predict the temperature profiles in timber connections, proper thermal material properties and furnace boundary conditions should be defined in the heat transfer model. Validation of the heat transfer model shows that adopting the temperature-dependent properties of wood and steel from Eurocode 5 as well as the temperature provided by the thermocouples and the plate thermometers to define

the thermal boundary conditions leads to a good agreement with full-scale test results.

15- The validations of the thermal and structural analysis numerical models show that the models predicted the failure time of the CN, the EX and the SE connections within a range of ± 1.5 -15 % .

16- The proposed empirical formulas can be used as a basic design tool to calculate the failure time of the CN, the EX and SE connections to design the connections to satisfy the required fire resistance by the building code.

6.3 Recommendations for Future Work

The following are recommendations for future studies on the fire performance of hybrid timber-steel connections.

- 1- In order to improve the accuracy of the proposed numerical formula for calculating the failure time for design purposes, further studies involving more specimens for each type of connection is recommended.
- 2- In this study the partial protection provided by the steel plate in the EX and SE connections improved the fire performance of these connections compared to the CN connections. Further studies are recommended to investigate the impact of additional fire protection such as the use of gypsum board, countersunk bolts, and intumescent paint.
- 3- It was observed that the number and the diameter of the bolts had a noticeable impact on the failure time of the connections. Therefore, further studies involving

the use of a bolt with a diameter smaller than 12.7 mm or replacing the bolts with dowels is recommended.

- 4- In the CN connection the 9 mm plate placed in the beam's centre notch transmits heat to the core of the beam and reduces the failure time. Further studies involving a 6 mm thick plate or multiple 4 mm thick plates to investigate the impact of plate thickness on fire performance of the CN connections is recommended.
- 5- In this study one size and one span of glulam beam was used. Additional studies are recommended to better understand the effect of other parameters such as beam span, wood side member thickness, beam effective depth and the different types of wood.
- 6- The approach adopted in the ABAQUS to model the wood was the Linear Elastic Fracture Mechanics method, which accommodated only linear elastic materials, and did not include the plastic deformation of the wood. Further studies incorporating the Concrete Damage Plasticity (CDP) model in ABAQUS is recommended to improve the model.
- 7- Further studies should be made to come up with connection configurations to achieve the 60 min and 90 min fire resistance rating required by codes.
- 8- The validated model can be used to study different parameters such as beam span, beam size, different type of wood and different size of bolts and steel plate thickness; however more full-scale test should also be conducted for additional model validation.

Appendix

Typical ambient design calculation as per CAN/CSA-O86-01 [36] for the Concealed connections with 4-19.1 mm bolts are presented in this Appendix.

The plate and beam section details are shown Figure A a and b and details calculation in shown for beam shear, bending and connection capacity are shown below.

a) Concealed plate details

b) Beam and bolt details

Figure 228 Concealed plate details

a=	600	mm	d(wood)=	191	mm	Density=	5.5	kN/m ³
c=	700	mm	b(wood)=	140	mm	D.L =	0.279433	kN
L _u =	1900	mm	e _p =	45	mm			
P=		kN	S _Q =	60	mm			
M= jP =	0	kN-m	S _p =	65	mm			
V=P	0	kN	a=	80	mm			
E=	12800	Mpa	de= d-e _p	146	mm			

Design for shear				6.5.7 CSA 086-09				
1		Simplified method						
V _r = ØF _v (2/3)A _g K _N								
F _v = f _v (K _D K _H K _{sv} K _T)								
f _v =	2.2	Mpa	Nordic Spec					
K _D =	1.15	Table 4.3.2.2						
K _H =	1	Clause 6.4.3	An = Ag - Ar >= 0.75 Ag	Clause 6.5.4				
K _{sv} =	1	Table 6.4.2	Ar = nd x df x b	5348	mm2	=	20%	O.K
K _T =	1	Clause 6.4.4	An =	21392	mm2	=	80%	O.K
K _N =	1	Clause 6.5.7.2.2	nd = # of dowels in gross section					
Ø=	0.9							
A _g =	dXb=	26740	mm ²					
A _{ge} =	deXb=	20440	mm ²					
F _v =	2.53	Mpa						
V _r =	40.59132	kN	shear at other section along the beam using d					
V _{re} =	31.02792	kN	effective shear at connection using de					

2	Detailed method	
$V_r = \phi F_v (0.48) A_g K_N C_v Z^{-0.18}$		$\geq W_f$ (total applied load) = 2P
$C_v =$	2.67	Table 6.5.7.3
$Z = L_u \times b \times d =$	0.050806 m ³	using d
	0.038836 m ³	using de
$V_r =$	133.4173 kN	using d $\geq 2P$
$V_{re} =$	107.0371 kN	using de $\geq 2P$
		$V_r = P = 66.709$ kN
		$V_r = P = 53.519$ kN

Design for Bending	6.5.6 CSA 086-09
$M_r = \phi F_b S K_x$ (lesser of K_{zbg} and K_L)	
$F_b = f_b (K_D K_H K_{sb} K_T)$	

$f_b = 30.7$ Mpa Nordic Spec
 $K_{sb} = 1$ Table 6.4.2
 $K_{SE} = 1$ Table 6.4.2
 $F_b = 35.305$ Mpa
 $E = 12800$ Mpa Table 6.3 CSA 086-09
 $S = bd^2/6 = 851223.3$ mm³
 $\phi = 0.9$
 $K_x = 1$ Clause 6.5.6.5.2
 $K_{zbg} = 1.03(BL_u)^{-0.18} \leq 1$ B&L in m
 $K_{zbg} = 1.30725 > 1$

Use $K_{zbg} = 1$

To calculate K_L we need C_B & C_K Clause 6.5.6.4.4

$C_B = \text{SQR}(L_e X d / b^2)$ $L_e = 1.92 L_u$ Table 6.5.6.4.3

$C_K = \text{SQR}((0.97 E K_{SE} K_T) / F_b)$

if $C_B \leq 10$ $K_L = 1$
 if $C_K > C_B > 10$ $K_L = 1 - (1/3)(C_B/C_K)^4$
 if $50 > C_B > C_K$ $K_L = (0.65 E K_{SE} K_T) / (C_B^2 F_b K_x)$
 if $C_B > 50$ Change the section

$L_e = 3648$ mm

$C_B = 5.962331$

$C_K = 18.75309$

$K_L = 1$

$M_r =$	27.0472 kN-m
then $P =$	45.07866 kN

Design for Connection Yielding and Splitting Resistance 10.4.4.2 CSA 086-09

$n_s =$	2	number of shear plans
$n_f =$	4	number of dowels
$d_f =$	19.1	mm
$t_1 =$	64	mm
$t_2 =$	9.5	mm
$G =$	0.49	Table A.10.1
$f_u =$	450	Mpa Plate Grade 300W
$f_y =$	310	Mpa Bolt Grade A307
$\phi_{steel} =$	0.67	10.4.4.3.3.2
$\phi_y =$	0.8	10.4.4.3.3.2

For perpendicular to grain loading, joint should be designed in according with the following requirements

a $N_f \leq N_r$

N_f : factored load on the joint

N_r : factored lateral yielding resistance

c $Q_f \leq Q_r$

Q_f : factored load perpendicular to grain

$Q_r = Q_{S_{rT}}$

$Q_{S_{rT}}$: factored splitting resistance

1 Yielding Resistance 10.4.4.3 CSA 086-09

$N_r = \phi_y n_u n_s n_f$

$n_u =$ the smallest value of item a,c,d and g clause 10.4.4.3.2

$n_u^a = f_1 d_f t_1 =$	12.26 kN
$n_u^c = 1/2(f_2 d_f t_2) =$	102.58 kN
$n_u^d = f_1 d_f^2 \{ \text{SQRT} \{ (1/6) * (f_2 / (f_1 + f_2)) (f_y / f_1) \} + (1/5) (t_1 / d_f) \} =$	8.7935 kN
$n_u^g = f_1 d_f^2 \{ \text{SQRT} \{ (2/3) * (f_2 / (f_1 + f_2)) (f_y / f_1) \} =$	16.535 kN

$f_1 = f_{1Q} K_D K_{SF} K_T =$ 10.02917 Mpa

10.4.4.3.3.1 for side member wood

$K_{SF} =$ 1

Table 10.2.1.5

$f_{1Q} = 22 G (1 - 0.01 d_f)$ 8.72102 Mpa

$f_2 = 3 f_u (\phi_{steel} / \phi_y)$ 1130.625 Mpa

10.4.4.3.3.1 for main member steel

$N_r =$ 56.27872 kN

2	Splitting Resistance for wood 10.4.4.7 CSA O86-09
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$Q_r = Q_{S_{rT}}$ for wood member in the joints (we have 2 side members)

$Q_{S_{rT}} = \sum Q_{S_{ri}}$ summation of 2 side members

$\phi_w = 0.7$

$Q_{S_i} = 14 \text{ t SQRT}(d_e/(1-d_e/d)) =$	22.30462 kN
$Q_{S_{ri}} = \phi_w Q_{S_i}(K_D K_{SF} K_T) =$	17.95522 kN
$Q_{S_{rT}} =$	35.91044 kN
$Q_r = Q_{S_{rT}}$	35.91044 kN

SUMMARY				
BEAM		Total load 2P		
Vr simple=	1	40.59132 kN	81.18264 kN	
Vr detailed	2	66.70866 kN	133.4173 kN	
Vr = max of 1 & 2		66.70866 kN	133.4173 kN	
Mr =		27.0472 kN-m	90.15732 kN	
Joints				
Vr simple	1	31.02792 kN	62.05584 kN	
Vr detailed	2	53.51853 kN	107.0371 kN	
Vr = max of 1 & 2		53.51853 kN	107.0371 kN	
Nr (Yielding)		56.27872 kN	112.5574 kN	
Qr (Splitting)		35.91044 kN	71.82088 kN	

Total load due to

Splitting 71.82088
 Bending 90.15732
 Joint shear 107.0371
 Beam shea 133.4173
 Yeilding 112.5574

Maximum capacity for the beam & joint = 71.82088 kN	Splitting Govern
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Loading Ratio	30%	21.54626 kN
	60%	43.09253 kN
	100%	71.82088 kN

Applied L.L = Capacity - D.L =	30%	21.26683 kN
	60%	42.8131 kN
	100%	71.54145 kN

Round loads

22
43
72
21.5
43
71.5

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