Blind Bolted Connections for Steel Hollow Section Columns in Low Rise Structures

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Synopsis

Hollow sections have become increasingly popular in structural applications due to their superior load bearing capacity and natural aesthetic appeal. However, their use in low rise residential and commercial structures is hampered by the limited means of providing fully bolted site connections. This limitation can be overcome by using single sided or blind bolting. The merit of blind bolts is that installation requires access to one side only. This eliminates both the problem of lack of access and also the need to weld various accessories to the hollow sections.

This research project focuses on the development of a range of connections to hollow sections using blind bolts. A series of tests have been performed to determine the behaviour of various configurations of blind bolted connections, exploring connections to the (i) face of the hollow section column, (ii) side wall of the column and (iii) front and back face of the column. This has resulted in the development of blind bolted moment connections for hollow section columns without concrete infill.

Comprehensive three-dimensional finite element (FE) models have been developed to predict the behaviour of the developed connection with sufficient accuracy. Extensive parametric studies have been conducted to explore the effects of various parameters on the performance of these connections with specific focus on the stiffness. Simplified theoretical models based on the component model
method proposed by Eurocode 3 have been formulated. These provide a quick and easy way for practising engineers to determine the stiffness of the proposed connections. Both the FE and theoretical models have been thoroughly validated against the experimental results.

The successful development of the proposed blind bolted moment connections offers a convenient and efficient alternative for beam-to-hollow section column connections in low rise structures.
Declaration

The work contained within this thesis is the original work of the author under the direction of the author’s supervisors and industrial sponsors. To the author’s best knowledge, this thesis contains no material previously published or written by another person except where cited in the text. This thesis has not been submitted to any institution other than the University of Melbourne for the degree of Doctor of Philosophy. This thesis is less than 100,000 words in length, exclusive of figures, tables, references and appendices.

Jessey Lee
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I would like to take this opportunity to express my utmost appreciation to both my supervisors, Dr. Helen Goldsworthy and Professor Emad Gad for their invaluable support, encouragement and guidance. I am grateful for their dedication, commitment and assistance which deeply motivated and inspired me. I have learnt so much from both my supervisors not only in terms of acquiring many analytical and critical thinking skills that are essential for a research project but also professional development in terms of communication, presentation and management skills. The three and a half years spent on this project with both of them have been most valuable, rewarding and a wonderful experience.

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Table of Contents

Synopsis ..............................................................................................................i
Declaration .......................................................................................................iii
Acknowledgements ...........................................................................................iv
Table of Contents ..............................................................................................vi
List of Figures ...................................................................................................xii
List of Tables ...................................................................................................xxi
Notation ....................................................................................................... xxiii
Preface ......................................................................................................... xxvii

Chapter 1
Introduction .......................................................................................................1

1.1 Background ................................................................................................1
1.2 Research Objectives ...............................................................................5
1.3 Research Methodology ..........................................................................6
1.4 Overview of Thesis ..............................................................................8

Chapter 2
Literature Review ...........................................................................................11
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Introduction</td>
<td>11</td>
</tr>
<tr>
<td>2.2</td>
<td>Single sided or blind bolting systems</td>
<td>12</td>
</tr>
<tr>
<td>2.2.1</td>
<td>Flowdrill</td>
<td>12</td>
</tr>
<tr>
<td>2.2.2</td>
<td>Huck Blind Bolt</td>
<td>14</td>
</tr>
<tr>
<td>2.2.3</td>
<td>Lindapter Hollobolt insert</td>
<td>15</td>
</tr>
<tr>
<td>2.2.4</td>
<td>Ajax ONESIDE</td>
<td>16</td>
</tr>
<tr>
<td>2.3</td>
<td>Classification of joints</td>
<td>21</td>
</tr>
<tr>
<td>2.4</td>
<td>Current design practice</td>
<td>29</td>
</tr>
<tr>
<td>2.4.1</td>
<td>Simple shear connections</td>
<td>29</td>
</tr>
<tr>
<td>2.4.2</td>
<td>Moment connections</td>
<td>32</td>
</tr>
<tr>
<td>2.5</td>
<td>Stiffening methods</td>
<td>38</td>
</tr>
<tr>
<td>2.5.1</td>
<td>Face connection</td>
<td>39</td>
</tr>
<tr>
<td>2.5.2</td>
<td>Side shear connection</td>
<td>51</td>
</tr>
<tr>
<td>2.5.3</td>
<td>Summary</td>
<td>59</td>
</tr>
</tbody>
</table>

Chapter 3

Preliminary Work | 61 |
| 3.1 | Introduction | 61 |
| 3.2 | Study of bolt pull-out behaviour | 62 |
| 3.3 | Preliminary tube model | 65 |
| 3.4 | Bolt pretension | 70 |
| 3.5 | Summary | 78 |

Chapter 4

Blind Bolted T-stub Connection | 80 |
| 4.1 | Introduction | 80 |
| 4.2 | Design concept | 81 |
4.3 Experimental program .............................................................. 82
  4.3.1 Test specimens ................................................................. 82
  4.3.2 Experimental setup and instrumentation ......................... 85
4.4 Experimental results .............................................................. 87
  4.4.1 T-stub strength and failure modes ................................. 87
  4.4.2 T-stub stiffness ............................................................... 92
4.5 Finite element (FE) analysis .................................................. 94
  4.5.1 Contact elements and material properties ....................... 94
  4.5.2 Comparison of FE predictions to experimental results .... 97
  4.5.3 Sensitivity analyses ....................................................... 104
  4.5.4 Modeling of overall T-stub connection ......................... 107
4.6 Design model ........................................................................ 112
  4.6.1 Column face plastification – strength ............................ 114
  4.6.2 Component model – initial stiffness ............................... 117
4.7 Summary and conclusions .................................................... 122

Chapter 5
Blind Bolted Collar Plate Connection ........................................... 125
  5.1 Introduction ........................................................................ 125
5.2 Experimental program ........................................................ 126
  5.2.1 Test specimen ............................................................... 126
  5.2.2 Experimental setup and instrumentation ....................... 127
5.3 Test results ........................................................................... 128
  5.3.1 Collar plate connection strength and failure mode ........ 128
  5.3.2 Deformation of collar plate ............................................ 129
  5.3.3 Relative movement between tube side wall and collar side plate 130
  5.3.4 Strains ........................................................................ 131
  5.3.5 Comparison of collar plate connection to T-stub connection 131
5.4 Finite element analysis ......................................................... 133
5.4.1 Comparison between the FE analysis and test results .......... 133
5.4.2 Sensitivity analysis ...................................................... 135
5.4.3 Moment rotation curves – comparison to EC3 specifications ... 138
5.5 Summary and conclusions ................................................. 140

Chapter 6
Connection to sides of hollow section columns ......................... 142

6.1 Introduction ........................................................................ 142
6.2 Experimental program ...................................................... 144
   6.2.1 Test specimen .......................................................... 144
   6.2.2 Experimental setup and instrumentation ......................... 144
6.3 Test results ................................................................. 146
   6.3.1 Overall beam deformation .......................................... 148
   6.3.2 Relative moment between beam and channel .................. 149
   6.3.3 Relative moment between tube and side plate .................. 150
   6.3.4 Relative moment between tube and side plate .................. 151
   6.3.5 Moment rotation response .......................................... 153
6.4 Finite element analysis ..................................................... 155
   6.4.1 Comparison to test results .......................................... 157
   6.4.2 Sensitivity analysis .................................................... 161
6.5 Design models ............................................................... 168
   6.5.1 Strength ................................................................. 168
   6.5.2 Serviceability .......................................................... 168
6.6 Summary and conclusions ................................................. 176

Chapter 7
Extended T-stub connection with back face support ...................... 178
Chapter 9

Conclusions and Recommendations ................................................................. 231

  9.1 Summary and contributions ................................................................. 231
  9.2 General conclusions ........................................................................... 232
  9.3 Recommendations for further study .................................................... 236

References........................................................................................................... 239

Appendix A

Proposed strength design of T-stub connections for SHS columns ............... 247

Appendix B

Strength design procedure for channel side plate connection with Ajax ONESIDE ................................................................. 258

Appendix C

Strength design procedure for blind bolted simple connections with Ajax ONESIDE ................................................................. 264
List of Figures

Figure 1-1: Examples of hollow section applications as tubular trusses ................. 2
Figure 1-2: Moment connection for Duragal House Frame (courtesy of Australian Tube Mills) ........................................................................................................ 3
Figure 1-3: T-stub connection with cogged extensions on blind bolts (H. Yao et al., 2008) ............................................................................................................... 4
Figure 1-4: Research framework .............................................................................. 7
Figure 2-1: Flowdrill process (Yeomans, 1996a) .................................................. 13
Figure 2-2: Huck Ultra-Twist bolt before and after installation (Tabsh & Mourad, 1997) ............................................................................................................... 14
Figure 2-3: Lindapter Hollobolt (Lindapter) .......................................................... 16
Figure 2-4: Hollobolt and RMH (Barnett, 2001) ..................................................... 16
Figure 2-5: Ajax ONESIDE bolt assembly and installation tool (Fernando, 2008) 17
Figure 2-6: Installation procedure of Ajax ONESIDE bolt (courtesy of Ajax Engineered Fasteners) ................................................................. 18
Figure 2-7: Reinforced telecommunication towers in the USA with ONESIDE (Fernando, 2008) ................................................................. 19
Figure 2-8: Classification of joints by stiffness (European Committee for Standardisation (CEN), 2005) ........................................................................... 22
Figure 2-9: Nondimensional Connection Classification (Bjorhovde et al., 1990) 23
Figure 2-10: Unified classification system (Nethercot et al., 1998) ................. 26
Figure 2-11: Classification System for sway frame (Hasan et al., 1998) .......... 28
Figure 2-12: Side plate or shear tab connection to hollow section column (Australian Institute of Steel Construction, 1996) .......................................................... 30
Figure 2-13: Design checks for simple blind bolted connection (British Steel, 2000) ....................................................................................................................... 32
Figure 2-14: Blind bolted extended endplate I-beam to RHS column connection (Mourad, 1994) ............................................................................................... 34
Figure 2-15: Endplate yielding with bolts failure (Mourad, 1994) ....................... 36
Figure 2-16: Endplate plastification (Kurobane et al., 2005) .............................. 37
Figure 2-17: Column face plastification yield line patterns (Kurobane et al., 2005) ....................................................................................................................... 37
Figure 2-18: Typical extended endplate connection and details of tested connection (Mourad, 1994) .............................................................................40
Figure 2-19: Moment rotation curves for specimens S5, S7 and C1 (Mourad, 1994) ........................................................................................................................ 41
Figure 2-20: Comparison of hollow section side wall deflections for specimens S5 and S7 (Mourad, 1994).................................................................................... 42
Figure 2-21: Extended endplate connection reinforced with welded angles (Tabuchi et al., 1994)....................................................................................... 43
Figure 2-22: Full scale connection assembly test specimen (Tabuchi et al., 1994) 43
Figure 2-23: Moment-rotation curve for H-400×200×8×13 beam (Tabuchi et al., 1994) .............................................................................................................. 44
Figure 2-24: Extended endplate details (France et al., 1999b)............................ 45
Figure 2-25: Moment rotation curves for extended endplates with varying SHS wall thicknesses (France et al., 1999b) ............................................................. 46
Figure 2-26: Moment rotation curves for comparison of SHS steel grades (France et al., 1999b) ................................................................................................... 47
Figure 2-27: Moment rotation curves for comparison of concrete filled effects (France et al., 1999a).................................................................................... 48
Figure 2-28: Deformation of T-stub connection to SHS tube (Barnett, 2001) ..... 50
Figure 2-29: Connection configurations by Elghazouli et al. (2009) ............... 50
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-30</td>
<td>Moment connection with double sided coped strap angles (Giroux &amp; Picard, 1976)</td>
</tr>
<tr>
<td>2-31</td>
<td>Moment rotation curves (Giroux &amp; Picard, 1976)</td>
</tr>
<tr>
<td>2-32</td>
<td>T-stiffener connection (Shanmugam, 1997)</td>
</tr>
<tr>
<td>2-33</td>
<td>Moment-rotation relationship for MT1-MT5 (Shanmugam, 1997)</td>
</tr>
<tr>
<td>2-34</td>
<td>Proposed RHS beam-to-column connection (Kumar &amp; Rao, 2006)</td>
</tr>
<tr>
<td>2-35</td>
<td>Moment-rotation curve for channel depth of 38mm (Kumar &amp; Rao, 2006)</td>
</tr>
<tr>
<td>2-36</td>
<td>Moment-rotation curve for channel depth of 75mm (Kumar &amp; Rao, 2006)</td>
</tr>
<tr>
<td>3-1</td>
<td>Test specimen by Ivanyi (2008)</td>
</tr>
<tr>
<td>3-2</td>
<td>Failure mode for 1 mm plate thickness (Ivanyi, 2008)</td>
</tr>
<tr>
<td>3-3</td>
<td>Finite element model (quarter symmetry)</td>
</tr>
<tr>
<td>3-4</td>
<td>Deformed shape</td>
</tr>
<tr>
<td>3-5</td>
<td>Comparison of experiment results (Ivanyi, 2008) with FE model</td>
</tr>
<tr>
<td>3-6</td>
<td>Simplified tube model, case A</td>
</tr>
<tr>
<td>3-7</td>
<td>Case B - stiffened with top plate</td>
</tr>
<tr>
<td>3-8</td>
<td>Case C - stiffened with side plates</td>
</tr>
<tr>
<td>3-9</td>
<td>Deformed shape of Case A, hollow section by itself</td>
</tr>
<tr>
<td>3-10</td>
<td>Maximum column face and corner stresses (Case A)</td>
</tr>
<tr>
<td>3-11</td>
<td>Deformation at middle cross section for cases B and C</td>
</tr>
<tr>
<td>3-12</td>
<td>FE model of bolt pretension</td>
</tr>
<tr>
<td>3-13</td>
<td>Contact pressure in MPa - bilinear &quot;all deformable&quot;</td>
</tr>
<tr>
<td>3-14</td>
<td>Load-displacement graphs for bolt shank</td>
</tr>
<tr>
<td>3-15</td>
<td>$\sigma_1$ vs. applied load for bolt shank</td>
</tr>
<tr>
<td>4-1</td>
<td>Proposed T-stub connection</td>
</tr>
<tr>
<td>4-2</td>
<td>Specimen detail (S1, S2, S3)</td>
</tr>
<tr>
<td>4-3</td>
<td>Ajax ONESIDE bolt assembly and installation tool (Ajax)</td>
</tr>
<tr>
<td>4-4</td>
<td>Stress-strain curves from coupon tests (specimens A1, A2 and A3)</td>
</tr>
<tr>
<td>4-5</td>
<td>Specimen setup</td>
</tr>
</tbody>
</table>
Figure 4-6: Instrumentation for Specimen S2, location of bolt gauges (Kyowa) shown on plan view marked with bolded “x” .............................................................. 87
Figure 4-7: Deformed shape of specimen S2 at high level of applied load........ 88
Figure 4-8: Deformed shape of tube, endplate and bolts after failure (specimen S1) ....................................................................................................................... 89
Figure 4-9: Specimen S2 at failure (southern bolts pulling out from tube face) .... 90
Figure 4-10: Failure of tube face (punching failure of bolt holes at southern end) – specimen S2 ..................................................................................................... 90
Figure 4-11: Specimen S3 at failure .................................................................... 91
Figure 4-12: Crushing of side walls directly underneath the endplate (specimen S3) ................................................................................................................. 91
Figure 4-13: Comparison of endplate displacement between specimens S1 and S2 ................................................................................................................. 93
Figure 4-14: Endplate displacement for specimen S3 ........................................ 93
Figure 4-15: Full FE model of specimen .......................................................... 95
Figure 4-16: Contact surfaces in FE model ....................................................... 96
Figure 4-17: Comparison of FE model and LVDT readings for endplate deformation (specimen S1) ................................................................. 97
Figure 4-18: Comparison of FE model and LVDT readings for endplate deformation (specimen S2) .................................................................................... 98
Figure 4-19: Comparison of FE model and LVDT readings for endplate deformation (specimen S3) ................................................................. 99
Figure 4-20: Comparison of results for endplate longitudinal deformed profile between FE model and photogrammetry (Specimen S1) ...................... 101
Figure 4-21: Comparison of results for side wall deformed profile between FE model and photogrammetry at different load levels (Specimen S1) ............. 102
Figure 4-22: Comparison of deformation at corner of tube side wall between FE model and photogrammetry results (Specimen S3) ...................... 103
Figure 4-23: Comparison of strains in bolts between FE and bolt gauges (Specimen S2) .................................................................
Blind Bolted Connections for Steel Hollow Section Columns

Figure 4-24: Sensitivity analysis for endplate thicknesses ........................................ 105
Figure 4-25: Sensitivity analysis for sleeves’ bolt hole tolerance, 10mm endplate
................................................................................................................................. 106
Figure 4-26: Sensitivity analysis for coefficient of friction, $\mu$ between contact
surfaces .................................................................................................................... 107
Figure 4-27: FE model of full beam-to-column T-stub connection (elevation view)
............................................................................................................................... 110
Figure 4-28: Comparison of stiffness between full and partial FE model .......... 111
Figure 4-29: Moment rotation curve for 150×6 SHS, full T-stub connection ...... 111
Figure 4-30: Summary of T-stub connection design ........................................... 113
Figure 4-31: Column face plastification (Mourad, 1994) .................................... 114
Figure 4-32: Idealisation of yield line pattern from FE model from principal stress
$\sigma_1$ (MPa) ......................................................................................................... 117
Figure 4-33: Component model for tension region ........................................... 118
Figure 4-34: Component model for T-stub connection ..................................... 119
Figure 5-1: Details of the test specimen and instrumentation............................. 126
Figure 5-2: Collar plate connection specimen setup ......................................... 127
Figure 5-3: Buckling of tube bottom face at failure, localised bearing around bolt
hole ....................................................................................................................... 129
Figure 5-4: Load vs. displacement for collar plate ............................................ 130
Figure 5-5: Relative moment between collar side plate and tube side wall ........ 131
Figure 5-6: Strain at underside of tube ............................................................. 132
Figure 5-7: Comparison of load displacement curve for collar plate connection
and T-stub connection .......................................................................................... 132
Figure 5-8: 3-dimensional FE model (half symmetry) ....................................... 133
Figure 5-9: Comparison of FE and photogrammetry results for collar plate
displacement ....................................................................................................... 134
Figure 5-10: Comparison of photogrammetry and FE results for collar plate
longitudinal profile ............................................................................................. 135
Figure 5-11: Sensitivity analysis between 10mm and 20mm collar plate .......... 136
Figure 5-12: Comparison of load-displacement curves for different sleeve tolerance .............................................................................................................................. 137
Figure 5-13: FE model of full beam-to-column collar plate connection .......... 139
Figure 5-14: Moment rotation curve for collar plate connection from FE model 139
Figure 6-1: Procedure to assemble connection ................................................. 143
Figure 6-2: Photo of specimen and zoomed in view of connection ................. 145
Figure 6-3: Test specimen setup (dimensions are in mm) ................................. 145
Figure 6-4: Instrumentation for test specimen .................................................. 146
Figure 6-5: Deformed shape of specimen at failure ........................................... 148
Figure 6-6: Bearing of bolt holes for tube and channel ..................................... 148
Figure 6-7: Load vs. displacement for LVDT 1 – overall beam deflection, $\Delta_X$ (refer to Figure 6-4 for locations of LVDTs) .............................................................. 149
Figure 6-8: Load vs. displacement for LVDT 2 – beam to channel, $\Delta_Y$ .......... 150
Figure 6-9: Load vs. displacement for LVDTs 3 and 4 – tube to side plate, $\Delta_Y$ 151
Figure 6-10: Load vs. displacement for LVDTs 5 and 6 – beam flange to tube top face, $\Delta_Y$ .......................................................................................................... 152
Figure 6-11: Load vs. displacement for LVDTs 7 and 8 – beam web to tube side face, $\Delta_Y$ .......................................................................................................... 152
Figure 6-12: Load vs. displacement for LVDT 9 – beam web to tube top face, $\Delta_X$ .......................................................................................................... 153
Figure 6-13: Moment rotation response from LVDTs 7 and 8 ...................... 154
Figure 6-14: Comparison of deformation limits between collar plate and channel side plate deformation (LVDT 5) ................................................................. 155
Figure 6-15: FE model of side plate connection which simulates the experimental setup .............................................................................................................. 157
Figure 6-16: Comparison of FE and experiment for LVDT 1 – overall beam deflection ........................................................................................................... 158
Figure 6-17: Comparison of FE and experiment for LVDTs 3 and 4 – tube to side plate .......................................................................................................... 159
Figure 6-18: Comparison of FE and experiment for LVDTs 5 and 6 – beam flange to tube top face ................................................................. 159
Figure 6-19: Comparison of FE and experiment for LVDTs 7 and 8 – beam web to tube side face ................................................................. 160
Figure 6-20: Comparison of FE and experiment for LVDT 9 – beam to tube (horizontal) ................................................................. 160
Figure 6-21: Sensitivity analysis – varying coefficient of friction, $\mu$ ................. 162
Figure 6-22: Sensitivity analysis – vary bolt size ................................................. 163
Figure 6-23: Sensitivity analysis – vary side plate thickness................................. 164
Figure 6-24: Sensitivity analysis – vary tube thickness ......................................... 165
Figure 6-25: Sensitivity analysis – vary channel thickness .................................... 166
Figure 6-26: Sensitivity analysis – vary position of bolts for top and bottom connections ................................................................. 167
Figure 6-27: Comparison of moment rotation curves when bolts were offsetted from center of bolt holes ................................................................. 167
Figure 6-28: Summary of channel side plate connection design .......................... 169
Figure 6-29: Slip test setup with one plate sandwiched between two channels......... 172
Figure 6-30: FE model to determine bolt clamping stiffness ................................ 173
Figure 7-1: Test specimen setup (dimensions are in mm) .................................... 180
Figure 7-2: Instrumentation for test specimen .................................................. 181
Figure 7-3: Load vs. displacement for LVDT 1 – overall beam flange displacement, $\Delta_X$ (refer to Figure 7-2 for location of LVDT) ......................... 183
Figure 7-4: Load vs. displacement for LVDTs 2 and 3 – beam flange to T-stem, $\Delta_Y$ (refer to Figure 7-2 for locations of LVDTs) ......................... 183
Figure 7-5: Load vs. displacement for LVDTs 4 and 5 – T-stem to endplate, $\Delta_Y$ (refer to Figure 7-2 for locations of LVDTs) ......................... 184
Figure 7-6: Load vs. displacement for LVDTs 6 and 7 – beam web to endplate, $\Delta_Y$ (refer to Figure 7-2 for locations of LVDTs) ......................... 184
Figure 7-7: Deformation of tube face and channels at failure .............................. 185
Figure 7-8: Deformed shape of specimen at termination of test ....................... 186
Figure 7-9: Load vs. displacement for LVDTs 8 and 9 – channels in tension, $\Delta_y$ (refer to Figure 7-2 for locations of LVDTs) .......................................................... 188
Figure 7-10: Load vs. displacement for LVDT 10 – channel in compression, $\Delta_y$ (refer to Figure 7-2 for locations of LVDTs) .......................................................... 189
Figure 7-11: Moment rotation response of extended T-stub connection ............ 190
Figure 7-12: FE model of extended T-stub connection ...................................... 192
Figure 7-13: Comparison of FE and experiment for LVDT 1 – overall beam flange displacement .................................................................................................. 193
Figure 7-14: Comparison of FE and experiment for LVDTs 2 and 3 – beam flange and T-stem ..................................................................................................... 194
Figure 7-15: Comparison of FE and experiment for LVDT 6– beam web to endplate (tension) .......................................................... 194
Figure 7-16: Comparison of FE and experiment for LVDT 7– beam web to endplate (compression) .......................................................... 195
Figure 7-17: Comparison of FE and experiment for LVDTs 8 and 9 – channels in tension .................................................................................................. 195
Figure 7-18: Sensitivity analysis – vary channel thickness............................... 197
Figure 7-19: Sensitivity analysis – vary T-stub and backplate thicknesses ............ 198
Figure 7-20: Sensitivity analysis – vary tube thickness ....................................... 199
Figure 7-21: Summary of extended T-stub connection design ......................... 200
Figure 7-22: Component model for tension region .......................................... 203
Figure 7-23: Simplified channel model............................................................. 205
Figure 7-24: Simplified backplate model.......................................................... 208
Figure 8-1: Construction sequence for T-stub connection ............................... 215
Figure 8-2: T-stubs connection on all four directions ....................................... 216
Figure 8-3: Alternative connection in secondary direction using angle cleats ..... 217
Figure 8-4: Construction sequence for a combined site plate and T-stub connections .................................................................................................. 219
Figure 8-5: FE model of channel side plate and T-stub connection combination
Figure 8-6: Moment rotation curve for combination of channel side plate and T-stub connection ............................................................................................................. 220

Figure 8-7: Pinned connection in secondary direction ........................................... 221

Figure 8-8: Moment connection in secondary direction ......................................... 222

Figure 8-9: Construction sequence for extended T-stub connection ...................... 223

Figure 8-10: Moment connection in secondary direction ...................................... 224

Figure 8-11: Pinned connection in secondary direction ....................................... 225

Figure 8-12: Case study – influence of connection stiffness on overall frame stiffness (dimensions are in mm) ................................................................. 228
List of Tables

Table 2-1: Minimum RHS thickness for full tensile capacity of grade 8.8 Flowdrill system structural bolt (Yeomans, 1996a) ................................................................. 14
Table 2-2: Minimum RHS thickness for different sizes of HSBB ($f_y = 350$MPa) (Ghobarah, Mourad, & Korol, 1996) ................................................................. 15
Table 2-3: Advantages and limitations of AJAX ONESIDE ........................................ 20
Table 2-4: Design components of the extended blind bolted moment connection ....................................................................................................................... 35
Table 2-5: Schedule of tested connection (Mourad, 1994) ................................ 40
Table 2-6: Specimen details for tensile test (Tabuchi et al., 1994) ....................... 43
Table 2-7: Schedule of Flowdrill rigid joint tests (France et al., 1999b)................ 45
Table 2-8: Details of specimens (Giroux & Picard, 1976) ..................................... 52
Table 2-9: Details of specimens (Shanmugam, 1997) ......................................... 54
Table 3-1: List of simplified tube models .......................................................... 66
Table 3-2: Material properties for model ......................................................... 66
Table 3-3: Summary of results for simplified tube models .............................. 68
Table 3-4: Parametric analysis for bolt pretension ............................................. 72
Table 4-1: Details of specimens ..................................................................... 84
Table 4-2: Material properties for bilinear model ............................................. 97
Table 4-3: Assembly of component model (tension region) ......................... 120
Table 4-4: Comparison of component model to FE prediction of tension stiffness ....................................................................................................................... 121
Table 4-5: Comparison of T-stub connection stiffness in compression region to tension region ................................................................. 122
Table 6-1: Material properties for bilinear model (characteristic values) .......... 156
Table 6-2: Comparison of component model to FE prediction of tension region stiffness (150×150 SHS) ................................................................. 175
Table 6-3: Comparison of stiffness for tension and compression regions (150×150 SHS) .................................................................................... 175
Table 7-1: Material properties for bilinear model ........................................ 192
Table 7-2: Comparison of component method to FE prediction for tension region stiffness .............................................................................. 209
Table 7-3: Comparison of connection stiffness in compression region to tension region (FE model) ................................................................. 210
Table 7-4: Comparison of component method to FE prediction for overall connection stiffness ................................................................. 211
Table 8-1: Interstorey drift of frame for different connection stiffness .......... 229
Notation

\[ a \] = dimension of simplified channel flange
\[ A_c \] = bolt minor area
\[ a_e \] = edge distance from bolt centre-line to edge of plate
\[ A_g \] = gross area of cross section
\[ A_n \] = net area of cross section
\[ A_o \] = nominal shank area
\[ A_{plate} \] = cross sectional area of plate
\[ A_s \] = tensile area of bolt
\[ A_{stem} \] = cross sectional area of stem
\[ A_v \] = cross sectional area in shear
\[ b \] = dimension of simplified channel leg
\[ b_c \] = width of column
\[ b_{eff} \] = effective width of endplate
\[ b_{ep} \] = width of endplate
\[ d_b \] = diameter of bolt
\[ d_{bm} \] = centre to centre distance of top and bottom T-stem
\[ d_h \] = diameter of bolt hole
\[ E \] = Young’s modulus for steel, 200 000MPa
\[ F' \] = design force in T-stem
\[ f_{c,y} \] = column yield strength
\[ F_{ps} \] = bolt pull out resistance
\[ f_u \] = ultimate strength
Blind Bolted Connections for Steel Hollow Section Columns

\( f_{u,c} \) = ultimate strength of column
\( f_{ul} \) = minimum tensile strength of the bolt
\( f_y \) = yield strength
\( g \) = centre to centre distance between bolts on endplate (transverse)
\( G \) = shear modulus for steel 80000MPa
\( h_b \) = centre to centre distance between bolts on endplate (longitudinal)
\( I_b \) = second moment of inertia of beam,
\( I_{backplate} \) = second moment of area of backplate
\( I_{channel} \) = second moment of area for channel plate section
\( K_{backplate} \) = backplate in bending
\( K_{bc} \) = clamping of bolts between the beam and channels
\( K_{bolt} \) = blind bolt stiffness in tension
\( K_{channel} \) = stiffness of channel
\( K_{clamp} \) = clamping stiffness of bolts for beam flange and T-stem
\( K_{cp} \) = clamping of bolts between channel and side plates
\( K_{ep} \) = endplate bending stiffness
\( k_h \) = hole factor
\( K_{plate} \) = stiffness of plate extension
\( K_{pt} \) = clamping of bolts between side plates and tube side walls
\( k_i \) = a factor for eccentricity of loading
\( K_{tension} \) = overall stiffness of the joint in tension
\( K_{tube} \) = column stiffness
\( K_v \) = shear stiffness
\( l' \) = corner of tube to centre of bolt on channel
\( L_b \) = length of beam,
\( L_{bolt} \) = bolt elongation length
\( l_{ep} \) = length of endplate
\( L_{plate} \) = length of plate undergoing extension
\( L_{stem} \) = length of stem undergoing extension

\( m \) = distance from bolt centre-line to face of T-stem

\( M_{i, beam} \) = section capacity of beam

\( M_{i, column} \) = section capacity of column

\( n \) = number of bolt rows

\( N_{bb} \) = number of blind bolts

\( n_{ei} \) = No. of effective interfaces

\( N_{ep} \) = flexure capacity of endplate

\( n_n \) = No. of shear planes with threads included

\( N_{p1} \) = column face plastification load

\( N_{sb} \) = number of standard bolts

\( N_t \) = tension capacity

\( N_{tf} \) = bolt tension capacity

\( N_{nt} \) = minimum bolt tension at installation

\( n_x \) = No. of shear planes with threads excluded

\( p \) = bolt pitch

\( P_{bsc} \) = bearing capacity of SHS column wall

\( P_v \) = local shear capacity of the SHS column wall

\( Q \) = applied shear force on connection

\( R^* \) = reduction factor due to corner restraints

\( s_g \) = centre to centre distance between bolts on T-stem

\( t_{bh} \) = thickness of bolt head

\( t_c \) = thickness of column

\( t_{ep} \) = thickness of endplate

\( t_{nut} \) = thickness of nut

\( t_p \) = ply thickness

\( t_{stem} \) = thickness of T-stem

\( t_w \) = thickness of blind bolt washers

\( V_b \) = bolt bearing capacity

\( V_{b, bb} \) = blind bolt bearing capacity
\( V_{b, ib} \) = standard bolt bearing capacity
\( V_{ch} \) = channel capacity in shear
\( V_{ep, h} \) = horizontal shear capacity of endplate
\( V_{ep, v} \) = vertical shear capacity of endplate
\( V_f \) = bolt shear capacity
\( V_{f, bb} \) = blind bolt shear capacity
\( V_{f, sb} \) = standard bolt shear capacity
\( x \) = deformed length in shear
\( z \) = lever arm of the connection
\( Z_e \) = effective section modulus
\( \alpha_s \) = shear coefficient
\( \eta \) = column axial load reduction factor
\( \gamma_s \) = deflection coefficient
\( \mu \) = slip factor
\( \nu \) = Poisson’s ratio for steel, 0.3
\( \phi \) = capacity reduction factor
\( \sigma_{ep} \) = yield stress of endplate
Preface

Research outcomes from this PhD study have been published in three peer reviewed conferences and two international journals. Another journal paper is currently under review for publication. All these publications are listed below.

Chapter 4 has in part, been published and presented in the following papers:


Chapter 5 has in part, been published and presented in the following paper:

Chapter 6 has in part, been published and presented in the following paper:


Chapter 7 has in part, been published in the following paper:


Chapters 6 and 8 have in part, been published in the following paper:

Chapter 1

Introduction

1.1 Background

Hollow sections have become increasingly popular in structural applications due to their structural efficiency and natural aesthetic appeal for exposed members. Being a closed section, they have high torsional stiffness which is greatly beneficial for many applications. They also have large radius of gyration especially about the minor axis, thus are less prone to buckling and have superior load bearing capacity. Hollow sections have a higher load carrying capacity per weight ratio compared to open sections and hence could provide savings in material costs.

Hollow sections have been used widely in tubular trusses in large scale construction of stadia and bridges such as the curved roof trusses of Southern Cross Station in Melbourne (Figure 1-1a), the grand arch of Wembley Stadium in London (Figure 1-1b) and the 140m span tubular arch bridge in Logroño, Spain. In all these structures, the trusses have been prefabricated and welded off site for ease of construction.
Welding remains the common practice to construct hollow section connections. Welding on site is undesirable due to the need for highly skilled labour and weather constraints. Current practice in the more modest construction industry of low rise residential and commercial structures tends to opt for open section columns and beams due to the limited means of providing fully bolted site connections.

Currently, connections between I-beams to hollow section columns are achieved primarily via welding appendages like shear tabs and angles on the hollow section wall to connect to the I-beams. These are flexible connections transferring shear only from the beams to the hollow section columns. The current construction trend is towards longer floor spans with large column free spaces to increase lettable or living floor areas and add architectural and functional flexibility to the building. This requires beam to column connections with high stiffness to reduce deflections and possible vibrations while keeping the beams size constant due to head height limitations. However, the use of conventional welded moment connections with hollow section columns is very limited due to the complex nature of the design and installation. An example of a moment connection currently used in Australia is shown in Figure 1-2. This connection is designed by Australian Tube Mills and is part of the Duragal House Frame at Seven Mile.
Beach, New South Wales. The connection is achieved by shop welding angles to the side walls of the hollow section and the beams are then bolted to the angles. As with all welded connections, their configurations are restricted by the arrangements of pre-welded components and they have tight installation tolerances on site.

![Figure 1-2: Moment connection for Duragal House Frame (courtesy of Australian Tube Mills)](image)

The restrictions and limitations imposed by welding can be overcome by the development of a single sided or blind bolting system in structural applications. The blind bolts provide a major economic benefit by requiring installation from one side only. This eliminates both the problem of lack of access and also the need to weld various appendages to the hollow sections; thus facilitates easier fabrication, transportation and erection.

The objective of this project is to develop moment connections for hollow section columns in low rise structures using ONESIDE blind bolts (sometimes referred to as “ONESIDE”) developed by Ajax Engineered Fasteners in Australia. Earlier studies were initiated by Gardner & Goldsworthy (2005) and Yao et al. (2008) on
developing a moment connection suitable for medium rise building frames in low to medium seismicity regions. An innovative modification was made to the ONESIDE blind bolt by adding a cog bar to the bolt head. The cogged extension acted as an anchor in concrete filled circular hollow section (CHS) column as shown in Figure 1-3. This resulted in a dramatic increase in stiffness and strength of the connection.

![Figure 1-3: T-stub connection with cogged extensions on blind bolts (H. Yao et al., 2008)](image)

One of the major focus point of this specific project is to determine the design requirements for the application of ONESIDE in low rise structures in which the tubular columns, although small, have ample capacity to carry the low amount of load imposed on them. The low rise columns do not require composite action from concrete infill to increase their load carrying capacity. Conversely, columns in the moderate rise buildings will benefit from the increased capacity given by concrete infill as these columns carry greater loads. Without concrete infill, the connection is more flexible and achieving a rigid connection poses a significant challenge.

In response to these research needs, a collaborative research program which is supported by the Australian Research Council (ARC) has been undertaken
between three universities in Australia, the University of Melbourne, Swinburne University of Technology and University of Western Sydney and also industry partners Australian Tube Mills (ATM) and Ajax Engineered Fasteners.

1.2 Research Objectives

This research project focuses on the development of a range of connections to hollow sections using blind bolts. The emphasis is on the development of rigid blind bolted connections for square hollow section columns without concrete infill. The research broadly aims to achieve the following objectives:

1) Perform a detailed literature review to highlight current practice and investigate previous attempts to produce moment resisting connections to hollow sections, and also to identify existing guidelines for classification of connections.

2) Investigate key factors which may affect the performance of typical bolted connections, including bolt pretension, thickness of connection components, size of bolt holes and location of bolts within a tube face.

3) Develop concepts for new blind bolted connections utilising the innovative Ajax ONESIDE bolt. Perform testing on the developed connections to assess their strength and stiffness properties.

4) Develop detailed non-linear FE models for the proposed connections which can predict the overall behaviour under static loading. The models are to be validated against the experimental results.
5) Formulate analytical models for the proposed connections, specifically for design purposes, using the component method.

6) Study the application of the proposed connections for a typical steel frame building, provide suggestions for connection configurations in the secondary framing direction, and propose practical construction procedures for the connections.

1.3 Research Methodology

This research project has utilised both experimental and analytical methods as summarised in Figure 1-4. The first phase of the research involved an extensive literature review of the current knowledge on the development of blind bolted moment connections for unfilled hollow section columns. The critical review of the literature identified knowledge gaps in the area, focused the research objectives, and helped to structure the research to achieve the desired outcomes.

In the second phase, preliminary finite element models were constructed to validate experiments from the literature review. The close match between the theoretical prediction and the experimental results engendered confidence in the analytical approach being employed. Preliminary parametric studies were undertaken to determine the basic parameters that contribute towards the connection stiffness.

In the third phase an experimental program was devised. The program commenced with simple component tests which involved testing the connections for tension and compression separately and built to more complicated second tier tests which involved full beam to column configurations.
Having conducted an extensive experimental program, analytical models were developed in the fourth phase and validated against the experimental results. Extensive investigation led to confidence in the accuracy of the analytical models and subsequently, sensitivity analyses were carried out to investigate influence of various parameters on the performance of the connections.

In the final phase, other innovative configurations were proposed and investigated with the analytical model. Connections from the secondary direction...
into the columns were also suggested for each of the proposed blind bolted connections. A case study of a typical steel frame was carried out to investigate the effect of the connection stiffness on the overall strength and response of the steel frames.

1.4 Overview of Thesis

This thesis consists of nine chapters. Chapter 1 introduces the background of the project; it states the aim and objectives of this thesis and describes the research methodology.

Chapter 2 presents a literature review of the available blind bolting systems in the market, types of blind bolted connection which cover both simple and moment connections, methods of stiffening these connections and classification systems for connection stiffness. The available design procedures for the current blind bolted connections are also reviewed and knowledge gaps identified to define the scope of the research.

Chapter 3 identifies critical parameters that affect typical face connection stiffness. Preliminary validated finite element (FE) models were utilised to assess these parameters and validates the exploratory model with published results. The findings from Chapter 3 assist in determining the geometrical arrangement for the test specimens in Chapter 4.

Chapter 4 describes the test setup for full scale blind bolted T-stub connection tests subjected to tension and compression loadings. The specimens are fully instrumented to capture the strain variations and deformations on the tube wall and endplate. A detailed three dimensional FE model is also presented and validated against the experimental results. Parametric studies of various tube
thicknesses, endplate thicknesses and geometrical arrangements are conducted with the FE model. A simplified component model is constructed to predict the connection stiffness using an approach proposed by Eurocode 3. In this approach the connection is broken up into individual components and the stiffness of these components is represented by springs.

Chapter 5 describes an alternative connection configuration as part of the development program to develop a rigid connection. The collar plate connection is tested in the tension region and the behaviour of the connection in terms of strength and stiffness is compared to the T-stub connection in Chapter 4. A detailed FE model is developed and validated against the experimental results. Parametric studies to determine the influence of variables such as collar plate thickness, size of bolt sleeve and pretension effects are conducted.

Chapter 6 reports the results and analyses from a full scale beam to column connection assembly for a new channel side plate connection which evolved from the concepts explored in the collar plate connection in Chapter 5. A detailed FE model is validated with the experimental results and sensitivity analyses are carried out for different channel, plate and tube thicknesses, bolt size and coefficient of friction. A simplified component model to predict the connection stiffness is also presented.

Chapter 7 explores another new configuration, the extended T-stub connection with back face support which is a modification of the T-stub connection in Chapter 4. Experimental results are discussed and compared with the FE model predictions. Parametric studies and a theoretical model to predict connection stiffness are also presented.

Chapter 8 compiles the findings of the preceding four chapters, discusses their application in practice in terms of construction sequence and provide suggestions
for connection details in the secondary direction. A case study of a moment resisting frame is also considered with various connection stiffnesses.

Chapter 9 provides a summary of the thesis and recommendations for further work from this research.

Appendix A outlines the strength design procedure for a blind bolted T-stub connection using Ajax ONESIDE blind bolts.

Appendix B outlines the strength design procedure for a blind bolted channel side plate connection using Ajax ONESIDE blind bolts.

Appendix C outlines the strength design procedure for simple blind bolted connections in the secondary direction using Ajax ONESIDE blind bolts.
Chapter 2

Literature Review

2.1 Introduction

Moment resisting connections to structural hollow sections offer many benefits in terms of allowing for longer span beams and smaller beam sections under gravity loading and also improving the lateral stiffness of steel frames; hence reducing the number and sizes of cross bracings required under lateral loading. In order to develop such connections, it is necessary to develop an understanding of the behaviour of various components of the connection; including the interaction between the blind bolts and the tube. A critical review of published information on the types of blind bolted connections available and their behaviour is presented in this chapter. Knowledge gaps are identified from current relevant literature.

This chapter provides an introduction to blind bolting systems available in the current market, discusses methods to classify the stiffness of a connection, and provides a review of previous studies on blind bolted connections and ways to improve the stiffness of these connections. The literature review examines the current design practice for connections between open section beams and hollow
section columns, reviews the adequacy of current design procedures and identifies specific areas where further research is required.

### 2.2 Single sided or blind bolting systems

Single sided or blind bolting systems can be used when only one side of the connection is accessible. This is greatly beneficial for the connections between I beams and hollow section columns as they can now be designed and detailed in a similar manner to the conventional connections between open sections. This section of the thesis gives an introduction to some of the blind bolts available in the market with the main focus on Ajax ONESIDE.

#### 2.2.1 Flowdrill

The Flowdrill system is achieved using a tungsten carbide bit rotating at high speed, drilling into a thin steel plate without removal of material. Heat generated from friction softens the steel plate and displaces material locally to form a lobe on the inner face. No material from the steel plate is removed in the process. The hole is then threaded with roll tapping and a standard structural bolt can be used for the connection (Yeomans, 1996a). A schematic of the Flowdrill process is shown in Figure 2-1. Flowdrilling is not suitable for pre-galvanised material; however some of the commonly used hollow sections in the Australian market such as DURAGAL are pre-galvanised.
The Flowdrill system involves a fully threaded bolt holding components together without the use of any nuts. There is a possibility that stripping of threads will occur under high loads. However, without any stiffening to the hollow section column face, the flexible column face deformation will be the governing failure mode irrespective of the type of fasteners used (Kurobane, Packer, Wardenier, & Yeomans, 2005).

The minimum thicknesses of hollow sections required to achieve the full tension strength of grade 8.8 structural bolts are given in Table 2-1. Higher tube wall thicknesses are required for larger bolt sizes. It is worth noting that these thicknesses are greater than that of rectangular and square hollow sections commonly used in low rise structures (typically 2mm – 6mm), only the full strength of M16 or smaller bolts are likely to be achievable, and that is only if the upper range of hollow section thicknesses is used. For cases where the full tensile capacity of the bolt cannot be developed, the limiting factor will be the flexibility of the hollow section. The information provided in Table 2-1 is applicable to RHS/SHS with nominal yield strength in the range of 275 to 355MPa.
2.2.2 **Huck Blind Bolt**

There are two types of structural blind fasteners produced by HUCK, the high strength blind bolt (HSBB) and Ultra-Twist. Both fasteners have slightly different components, resistance mechanisms and methods of installation. The Ultra-Twist is the latest development by Huck Inc. and its components are shown in Figure 2-2.

![Huck Ultra-Twist bolt](image)

**Figure 2-2: Huck Ultra-Twist bolt before and after installation (Tabsh & Mourad, 1997)**

The minimum hollow section column flange thicknesses required to achieve the tensile strength of the Huck HSBB is specified in Table 2-2. A comparison with Table 2-1 shows that a HSBB requires an even thicker column face to achieve the full tensile strength of the bolt compared to that of the Flowdrill system. Most
applications in the low rise structures will not utilise hollow sections with such high thicknesses.

Table 2-2: Minimum RHS thickness for different sizes of HSBB ($f_y = 350\text{MPa}$) (Ghobarah, Mourad, & Korol, 1996)

<table>
<thead>
<tr>
<th>Bolt size mm</th>
<th>Minimum RHS thickness mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>10.4</td>
</tr>
<tr>
<td>20</td>
<td>13.4</td>
</tr>
<tr>
<td>22</td>
<td>14.8</td>
</tr>
</tbody>
</table>

2.2.3 Lindapter Hollobolt insert

The Hollobolt has either three parts or five parts depending on its size. The larger size bolts i.e. M16 and M20 are five part systems while the smaller bolts have only three parts. The preassembled unit consists of a standard grade 8.8 bolt with a threaded truncated cone at the end of a main body with spreadable legs. The five part system is shown in Figure 2-3.

The Hollobolt only requires two spanners to install; the bolt is inserted through a predrilled hole and is tightened with a torque wrench. The tightening draws the cone into the legs of the main body. The legs are splayed out, forming a secure fixing against pulling out (Kurobane et al., 2005). The five part system also features a collapse mechanism which maximises the clamping force and is suitable for a moment connection.

Barnett (2001) devised a new type of Hollobolt known as the Reverse Mechanism Hollobolt (RMH) in which the extending part is inverted to improve the clamping action of the Hollobolt (refer Figure 2-4). The standard Hollobolt was found by Barnett (2001) to be less efficient in terms of clamping compared to the Huck HSBB and the Ajax ONESIDE blind bolts.
2.2.4 Ajax ONESIDE

The ONESIDE is a relatively new system compared to other available blind bolting systems in the present market. As shown in Figure 2-5, the ONESIDE bolt comprises a circular bolt head (A), internal collapsible stepped washer (B), sleeve (C), external solid stepped washer (D) and nut (E). A special installation tool (F) is required to insert the blind bolt and tighten it.
Procedures to install the ONESIDE bolt are summarised in Figure 2-6. Initially the bolt assembly is placed on the installation tool. The bolt and the collapsible (folding) washer are then inserted through an oversized bolt hole; the collapsible washer is folded to clear the bolt hole. Then, the insertion tool is rotated to unfold the collapsible washer and pulled back so that the bolt head and collapsible washer are bearing on the inside face of the tube wall. The solid washer and nut on the external face of the tube are then pushed along the shaft of the tool and the nut is tightened.
The largest current application of ONESIDE is in reinforcing telecommunication towers made of thin walled hollow steel sections in the United States of America (Fernando, 2008). Stiffening ribs are externally blind bolted to the tower providing an effective means of strengthening the tower without disruptions to
the operation of the tower. Figure 2-7 shows a reinforced telecommunication tower with Ajax ONESIDE.

![Reinforced telecommunication towers in the USA with ONESIDE (Fernando, 2008)](image)

Figure 2-7: Reinforced telecommunication towers in the USA with ONESIDE (Fernando, 2008)

The advantage of ONESIDE is in its capability to achieve the full structural strength of a standard bolt. Other blind bolts in the present market cannot develop the full strength of standard bolts because they rely on friction or deformable devices (sleeves) to form a head on the inaccessible side. In order for the devices to deform sufficiently, they have to be made of lower strength material compared to other parts of the bolt, thus compromising the load carrying capacity of the bolt (Fernando, 2008). The ONESIDE does not rely on the deformability of any components, thus it operates like a standard bolt. It also has thicker washers compared to standard ones, providing additional rigidity in the fully clamped condition. The additional sleeve (part C in Figure 2-5) increases the shear capacity of ONESIDE compared to a standard bolt and compensates for the oversize hole.

However, it is worth noting that in a hollow section connection, the performance of the ONESIDE may be limited by the thickness of the tube wall. Previous testing by Gardner & Goldsworthy (2005) found that large localised deformation
occurred around the bolt hole of a 219.1 × 4.8mm CHS causing the bolt to be pulled out from the tube.

Another disadvantage is that the installation procedure of ONESIDE imposes geometrical limitations to the system. In order to get the bolt head and oversized washer through the bolt hole to the inaccessible side, an oversized bolt hole is required. Also, a slightly longer bolt is needed to accommodate both the oversized washers and the spigot. The most significant limitation is the minimum cavity length required to allow sufficient length for the bolt and collapsible washer to be inserted through the hole as shown in Figure 2-6c). Thus if the hollow section has a small depth, the installation of ONESIDE may not be possible. Table 2-3 summarises the advantages and limitations of Ajax ONESIDE.

<table>
<thead>
<tr>
<th>Advantages</th>
<th>Limitations</th>
</tr>
</thead>
<tbody>
<tr>
<td>▪ Easy to install</td>
<td>▪ Oversize bolt hole</td>
</tr>
<tr>
<td>▪ Can be designed according to AS 4100 like a standard bolt</td>
<td>▪ Minimum cavity length is needed to facilitate installation</td>
</tr>
<tr>
<td>▪ Able to achieve full tension capacity of grade 8.8 structural bolt</td>
<td>▪ Installation tool costly – unique to a particular bolt size</td>
</tr>
<tr>
<td>▪ Higher shear capacity than grade 8.8 structural bolt with oversized sleeve</td>
<td>▪ Requires larger minimum edge distance compared to a standard bolt</td>
</tr>
<tr>
<td>▪ Consistent capacity - does not rely on friction or deformable devices to achieve clamping force</td>
<td>▪ Not recommended for CHS with diameter smaller than 350mm due to curved profile of CHS</td>
</tr>
<tr>
<td>▪ Thicker washers compared to normal ones – higher rigidity when fully clamped</td>
<td>▪ Not suitable for plate thicknesses below 3mm – punching shear of bolt becomes critical (also applicable to other bolts)</td>
</tr>
</tbody>
</table>
2.3 Classification of joints

This project is directed towards the development of rigid blind bolted connections for hollow sections. A classification system is required to classify the connections proposed into their respective categories of stiffness. This section reviews some of the available classification systems and adopts one of the classification systems as the standard reference in this thesis.

The classifications of connections in AS 4100–Steel structures (Standards Australia, 1998) are made in general terms whereby the behaviour of connections in three forms of constructions (i.e. rigid, semi-rigid and simple) is described. However, there is no guidance given on how to evaluate the stiffness of a connection and to classify the connection according to its respective stiffness. The description under Clause 9.1.2 only mentions the expected degree of flexural restraint and joint deformation for each form of construction. There is no clear definition of the boundaries between the three types of connection stiffnesses.

A connection stiffness classification system is defined in Eurocode 3 part 1-8 (2005). Initial rotation stiffness, $S_{ini}$ for each form of construction is classified in Figure 2-8. This provides a straightforward method to identify the connection stiffness. It can be seen from Figure 2-8 that $S_{ini}$ is dependent on the connected beam properties. A larger beam requires a stiffer connection to achieve rigidity. It is worth noting that given the same beam properties, the stiffness required to achieving rigid behaviour in an unbraced or sway frame is around three times the stiffness required for a braced frame. It is the aim of this research to develop blind bolted connections between hollow section columns and open section beams that are at least stiff enough to be classified as rigid for braced frames. The stiffness classification by Eurocode 3 part 1-8 (2005) will be the main reference for discussions on connection stiffnesses in this thesis.
Prior to Eurocode 3 part 1-8 (2005), the classification system by Bjorhovde et al. (1990) followed by Eurocode 3 part 1-1 (1992) were the two main classification systems for beam to column connections. The classification system by Bjorhovde et al. (1990) as shown in Figure 2-9 is based on the reference length concept of the connected beam’s length to depth ratio, where $l_e$ is the reference beam length and $d$ is the beam’s depth. The system is suitable for cases where prior knowledge of the frame layout is unknown. The connection is considered in isolation from the frame and the classification may be inaccurate if the actual structural layout differs significantly from that considered by Bjorhovde et al. (1990).

On the other hand, the classification system by Eurocode 3 is based on the load carrying capacity of the frame and prior knowledge of members’ layout and sizes are required. The earlier version of Eurocode 3 part 1-1 (1992) is similar to
Eurocode 3 part 1-8 (2005), except it is based on non-dimensionalised parameters with reference to the plastic moment capacity of the connecting beam.

Goto & Miyashita (1995) and Kishi et al. (1997) reviewed the validity of the Eurocode 3 (1992) steel connection classifications. They both found that the Eurocode 3 classification is either conservative or sufficient in terms of ultimate strength classification. Carrying on from their earlier work to validate the Eurocode 3 classification system, Goto & Miyashita (1998) developed a new classification system to suit different connection types and location of joints in the frame. It has more stringent boundary limits compared to Eurocode 3 in achieving rigidity for both braced and unbraced frames. The researchers also specified the required rotational capacity for a connection to be classified as rigid which is not defined by Eurocode 3. The classification system by Goto & Miyashita (1998) involves different formulae to suit individual connection types and their location within the frame which may not be practical for design engineers.

On the other hand, Nethercot et al. (1998) developed a unified classification system whereby the stiffness and strength of the connections are considered simultaneously. This is to avoid confusion for design engineers in cases where a
connection may be rigid in stiffness but the full bending moment strength of the beam cannot be achieved, sometimes called the partial strength connection. Such cases are possible with the classification system by Eurocode 3 (2005), where stiffness and strength are classified separately. According to the unified classification system, connections can be placed into one of four categories:

- Fully-connected connections – full strength (moment capacity of connected beam can be developed) and rigid (sufficient stiffness to develop connection moment capacity)
- Partially-connected connections – connections that behave in between the fully-connected and pin-connected connections with sufficient ductility. Most connections fall into this category.
- Pin-connected connections – connections with low moment capacity and stiffness with sufficient ductility.
- Non-structural connections – connections that lack rotation capacity and fail prematurely.

The unified classification system places an explicit rotational requirement on connections which is not specified by the Eurocode 3 classification system. The system also considers stiffness requirements based on the column-to-beam rotational stiffness ratio of the connection. This is a more accurate classification compared to Eurocode 3 (2005) which assumes a typical column to beam stiffness ratio for all connections. Figure 2-10 shows the unified classification system for both the ultimate and serviceability limit states. Connection responses at both limit states are assessed. The ultimate limit states classification corresponds to mostly strength requirements while the serviceability limit states classification corresponds mainly to limiting deformation and hence the provision of stiffness.
The column-to beam rotational stiffness ratio, $\alpha$ is defined by Equation 2-1,

$$\alpha = \frac{\sum I_c/L_c}{I_b/L_b}$$

(2-1)

where:

- $I_c$ = second moment of inertia of column,
- $I_b$ = second moment of inertia of beam,
- $L_c$ = length of column,
- $L_b$ = length of beam,
- $M_b$ = moment in beam,
- $M_{p,b}$ = plastic moment of beam.
b) examples of applications of the unified classification system at ultimate limit state

![Diagram](image)

\[
\frac{M}{M_b} = \frac{(2+\alpha)(38\alpha)}{0.53-1/\alpha}
\]

\[
\frac{0.75/\alpha+0.37}{0.9} \quad \frac{\theta E l}{(M_b L)}
\]

\[
\text{fully-connected} \quad \text{partially-connected} \quad \text{non-structural}
\]

\[
\text{partially-connected} \quad \text{pin-connected}
\]

\[\text{(2+\alpha)(38\alpha)}\]

\[0.53-1/\alpha\]

\[0.75/\alpha+0.37\]

\[0.9\]

\[\theta E l/(M_b L)\]

\[\text{fully-connected zone}\]

\[\text{partially-connected zone}\]

\[\text{non-structural zone}\]

\[\text{pin-connected zone}\]

\[\frac{(20+12\alpha+\alpha^2)}{(70\alpha-20\alpha)}\]

\[\frac{1/6-1/\alpha}{1/3}\]

\[\frac{(7\alpha-2)}{(2\alpha)}\]

\[\frac{\theta E l}{(M_b L)}\]

c) unified classification system at serviceability limit state

Figure 2-10: Unified classification system (Nethercot et al., 1998)
Hasan et al. (1998) argued that the use of stiffness ratio between the connection and the beam adopted by Bjorhovde et al. (1990) (reference length, \( l_e = 2d \) or \( 10d \) in Figure 2-9) and Eurocode 3 (\( k_b = 25 \) for sway, 8 for braced and 0.5 for pinned in Figure 2-8) is inaccurate as this stiffness ratio varies with beam and frame properties. To improve the accuracy of both systems, Hasan et al. (1998) proposed a non-linear classification system for sway frames that does not rely on the beam stiffness. The classification shown in Figure 2-11 is based on three parameters, namely: initial stiffness of the connection \( R_{ki} \), ultimate moment capacity of connected members \( M_u \) (usually plastic moment capacity of beam) and shape parameter \( n \) to model the nonlinearity of the classification system. This model is expressed by Equation 2-2:

\[
M = \frac{R_{ki} \theta_r}{\left[1 + \left(\frac{\theta_r}{\theta_o}\right)^n\right]^\frac{1}{n}} \quad (2-2)
\]

where \( M \) = connection moment,
\( \theta_r \) = relative connection rotation,
\( \theta_o \) = reference plastic rotation = \( M_u / R_{ki} \),
\( R_{ki} \) = initial connection stiffness,
\( M_u \) = ultimate moment capacity of connection,
\( n \) = shape parameter.
Although there are several classification systems available, this thesis will adopt the Eurocode 3 classification system for consistency as it is relatively easy to use and has been shown to be conservative by several researchers (Goto & Miyashita, 1995; Kishi et al., 1997). Furthermore, it is expected that the serviceability condition will govern for the low rise connections developed in this project. Under serviceability condition, the connection remains approximately linear and can be represented by the initial stiffness slope. However, if a more accurate classification is required, the nonlinear classification system proposed by Hasan et al. (1998) for sway frames or the unified classification system by Nethercot et al. (1998) can be adopted after preliminary assessment based on Eurocode 3 (2005) has been made.
2.4 Current design practice

One of the objectives of the project is to develop design methodologies for the proposed connections. This section reviews current design procedures for hollow section connections and identifies the scope of additional work required in developing design guidelines for the proposed connections in this project.

2.4.1 Simple shear connections

In Australia, the design of connections for structural steel hollow sections is covered by the Australian Institute of Steel Construction, AISC (1996). The design guide focuses mostly on traditional welded connections between hollow section members. The only case covered by the design guide which involves connection between a hollow section column and I-beams is the side plate or shear tab connection shown in Figure 2-12. The side plate connection is designed such that yielding of the shear tab occurs before punching shear failure of the column face.

Another design guideline available from the AISC is by Hogan & Thomas (1994) which covers a number of standard connections for open sections. Simple connections such as angle seat, angle cleat, web site plate and rigid connections such as bolted endplate moment connection are covered by the design guide. Basic design checks for beams, connecting elements, bolts and welds that are covered in the design guide for connections to open section columns are also applicable to the same types of connections made to hollow section columns. However, additional design checks will be required for the hollow section column itself which are not currently covered by the Australian Standards or AISC design guidelines.
A more extensive coverage of design of hollow section connections can be found from research conducted in Europe whereby a large proportion of the work has been performed under the guidance of CIDECT (International Committee for the Research and Development of Tubular Construction). Various types of welded simple shear connections to hollow section columns (e.g. shear tab, single and double angle connections) are covered in the CIDECT Design Guide 9 (Kurobane et al., 2005). However, there is no direct guidance provided in the CIDECT Design Guide 9 for the design of simple shear blind bolted connections except it is stated that “previous testing of such connections using blind bolts has not shown any extraordinary failure modes compared to normal practice”. As the Ajax ONESIDE has a different structural geometry and resistance mechanism compared to the blind bolts tested previously, basic behaviour of the ONESIDE in tension and shear needs to be checked.

A collaborative effort between the Steel Construction Institute (SCI) and the British Constructional Steelwork Association Limited (BCSA) led to the publication of a design guide for simple connections which includes blind bolted double angle web
cleats and flexible endplate connections to hot finished RHS columns. Design tables are provided for the two types of blind bolts widely used in the UK i.e. Flowdrill and Hollobolt.

Checks for blind bolted simple connections based on recommendations from the Steel Construction Institute (SCI) & British Constructional Steelwork Association (BCSA) (2002) are illustrated in Figure 2-13. The checks comprise local shear and bearing capacity of the RHS column wall and bolts, tie force capacity of the RHS column wall in the presence of axial compression in the column, and structural integrity checks for bolt pull-out from the RHS column wall. The simple connection is designed to support shear loads from the beam reactions, axial forces in the connected beam and to accommodate some flexibility in beam rotation. The SCI/BCSA rules provide clearer design guidelines for blind bolted simple connections compared to the CIDECT Design Guide 9. However, the rules do not cater for blind bolted moment connections.

The current guidelines for blind bolted simple connections have adequately covered all of the design limit states. Hence it is possible to design a simple connection using the Ajax ONESIDE based on the design procedures given by the SCI/BCSA rules.
2.4.2 Moment connections

A moment connection is a connection joint designed to transfer bending moments from a beam to the connected column through the top and bottom flanges of the beam. The ends of a beam in a moment connection are restrained from rotating.
An example of a typical moment connection is shown in Figure 2-14. The extended endplate connection is made up of a beam fully welded to an endplate. The endplate is then either bolted or welded to the connecting column.

Design of blind bolted moment connections are covered in CIDECT Design Guide 9 summarising the work by several researchers (Occhi, 1996; Yeomans, 1994, 1996a, 1996b, 1998) on extended endplate blind bolted moment connections. In addition to the research on Flowdrill and Lindapter Hollobolt connection systems, extensive research has been conducted on the extended endplate connections with Huck HSBB by Ghobarah et al. (1996), Mourad (1994) and Mourad et al. (1995; 1996). This work was conducted in North America.

Figure 2-14 shows the details and design actions of a typical extended endplate connection. The connection is designed to transfer negative moments from the beam to the RHS column via tension and compression forces, $P_f$. In the tension region, the bolts are subjected to a pull-out force deforming the endplate and the flexible column flange. The column flange deforms outwards due to tension forces, induces rotations at the corners of the tube and causes the side walls to deform inwards. The inward curving of the side walls adds on to the overall flexibility and deformation of column flange.

Mourad (1994) suggested that when wind load governs for a structure, the connection can be designed to carry the plastic moment of the attached beam, $M_{design} = M_{p, beam}$. However if earthquake load governs, the design moment of the connection should be 1.3 times the attached beam plastic moment, $M_{design} = 1.3 M_{p, beam}$ to allow the connection to behave elastically while energy is being dissipated in the beam’s plastic hinges.
The endplate is designed such that it fully yields before failure of the blind bolt occurs to ensure a ductile failure. However, the endplate should also be thick enough to reduce prying action in the bolts due to endplate deformation. For good detailing, dimension “a” should be equal to or greater than “b” in Figure 2-14 to reduce prying action in the bolts (Mourad, 1994).

Table 2-4 provides a list of design components for the extended endplate connection. The components are divided into three categories, namely fasteners (bolts in shear, bearing and tension, welds between I-beam to endplate), endplate and the hollow section column walls. The scope of design work covered by various researchers are also summarised in the table. A tick indicates that the design criterion has been considered by that particular researcher or researchers while a cross indicates that this criterion has not been reported on.

Design formulae for the fasteners and endplate are similar to a standard connection although consideration needs to given to the slightly different properties of the blind bolts. Fernando (2008) has suggested that the Ajax
ONESIDE can be designed according to the Australian Steel Standard AS4100 as long as the appropriate bolt properties are assumed.

Table 2-4: Design components of the extended blind bolted moment connection

<table>
<thead>
<tr>
<th>Design Components</th>
<th>Flowdrill (Yeomans, 1996b)</th>
<th>Huck HSBB (Mourad, 1994)</th>
<th>Hollobolt (Occhi, 1996)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Fasteners</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bolts in shear</td>
<td>✓</td>
<td>×</td>
<td>✓</td>
</tr>
<tr>
<td>Bolts in bearing</td>
<td>✓</td>
<td>×</td>
<td>✓</td>
</tr>
<tr>
<td>Bolts in tension</td>
<td>✓</td>
<td>×</td>
<td>✓</td>
</tr>
<tr>
<td>Welds</td>
<td>×</td>
<td>✓</td>
<td>×</td>
</tr>
<tr>
<td><strong>Endplate</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Endplate plastification</td>
<td>×</td>
<td>✓</td>
<td>×</td>
</tr>
<tr>
<td><strong>RHS</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Min. thickness to develop full tensile strength of bolt</td>
<td>Recommendations provided</td>
<td>✓</td>
<td>×</td>
</tr>
<tr>
<td>RHS thread shear</td>
<td>✓</td>
<td>Not applicable</td>
<td></td>
</tr>
<tr>
<td>RHS punching shear</td>
<td>✓</td>
<td>×</td>
<td>✓</td>
</tr>
<tr>
<td>RHS face yielding</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>RHS web bearing</td>
<td>✓</td>
<td>×</td>
<td>✓</td>
</tr>
</tbody>
</table>

Among the three researchers listed in Table 2-4, Mourad (1994) is the only one who investigated the endplate design. Mourad (1994) proposed a formula to determine the thickness of endplate required such that the endplate will yield before failure of the HSBB. This formula also considers the effects of the bolt hole and is given by Equation 2-3:

\[
t_{ep} \geq \sqrt{\frac{P_f b}{\Phi f_{y,c} (B_{ep} - d_h)}}
\]  

(2-3)

where \( t_{ep} \) is the endplate thickness, \( P_f \) is the design force in beam flange, \( b \) is the distance from the centre line of the bolts to the centre line of the beam flange.
Blind Bolted Connections for Steel Hollow Section Columns

Fillet weld, $\Phi$ is the resistance factor of 0.9, $f_{yw}$ is the yield strength of hollow section, $B_{ep}$ is the endplate width and $d_h$ is the diameter of bolt hole.

The failure mechanism of the endplate proposed by Mourad (1994) is shown in Figure 2-15. Having determined a suitable endplate thickness from Equation 2-3, the beam flange force $F_f$ that causes the mechanism in Figure 2-15 to occur is calculated from Equation 2-3:

$$F_f = \frac{t_{ep}^2 \sigma_{ep} B_{ep} + 8 T_{min} a}{2(a + b)}$$

(2-3)

where $T_{min}$ is the minimum tensile strength of the HSBB and $\sigma_{ep}$ is the yield stress of endplate. The design force in the beam flange $P_f$, is then checked such that $P_f$ is lesser than or equal to $F_f$ ($P_f \leq F_f$).

Due to the oversized bolt hole required for Ajax ONESIDE, verification is needed before the endplate design formulae can be applied directly. Figure 2-16 shows the modes of failure for an endplate depending on its stiffness; for example, when the endplate is very stiff, the bolt will fail without any yielding occurring in the endplate.
The next design step is the design of the hollow section column flange. Table 2-4 provides a list of various failure modes associated with the hollow section connection and at least one researcher out of the three listed has worked on each of these design criteria. However, adjustments to the current available formulae are necessary to suit Ajax ONESIDE as it has a different resistance mechanism and structural components from other blind bolts. Figure 2-17 shows the column face yield line patterns; the inherent flexibility of the column face likely to make it the weakest link in the connection.
Overall it can be said the ultimate limit states design for the extended endplate blind bolted connection is well covered and the present available formulae can be adopted for ONESIDE with some modifications and verification. However at present, there is no information available on determination of stiffness for this type of connection, hence it is not possible for design engineers to adopt semi rigid properties in the analysis.

Currently, the only way to determine the stiffness of an extended endplate blind bolted connection is via experimental tests to determine the moment-rotation relationship of the connection. Ghobarah et al. (1996) succeeded in developing a moment-rotation relationship for the extended endplate connection based on a four parameter exponential model. However finite element analysis is required to determine some of the parameters for the analytical model. Design charts and tables provided by Ghobarah et al. (1996) based on the conducted finite element analyses only cover a small range of SHS sizes and may not be robust enough for the different sizes of hollow section adopted in a design practice.

2.5 Stiffening methods

Discussions on previous studies of blind bolted connections and ways to improve their performance are covered in this section. The discussions in this section are focused on the inherent flexibility of the column face. Two main types of connections are considered, each working on a different type of force transfer mechanism between the beams and the hollow section columns.
2.5.1 *Face connection*

The term “face connection” in this thesis refers to beams connected to the face of the hollow section columns. One such example is the extended endplate connection which is the most common type of blind bolted moment connection. Several researchers have investigated the behaviour of such a connection with different types of blind bolts. Mourad (1994), Mourad et al. (1995; 1996), Ghobahar et al.(1996) and Tabuchi et al. (1994) were among the early researchers to investigate the behaviour of blind bolted moment connections. The scope of their coverage differed in connection size and functionality. Mourad (1994) studied the behaviour of typical extended endplate connections shown in Figure 2-18 while Tabuchi (1994) investigated the behaviour of moment resisting seismic connections which have more stringent strength and serviceability requirements compared to the connections investigated by Mourad (1994).

Mourad (1994) performed tests on beam-column joints with different configurations to examine the stiffness of blind bolted extended endplate connections. Moment rotation curves for the three cases considered by Mourad (1994) are listed in Table 2-5 and plotted in Figure 2-19. Stiffness boundaries based on Eurocode 3 part 1-8 (2005) classifications for a beam length of 7.5m are drawn on the graph to classify the connection stiffnesses. For consistency, all Eurocode 3 classification boundaries plotted in this chapter are based on an assumed beam length of 7.5m.
Figure 2-18: Typical extended endplate connection and details of tested connection (Mourad, 1994)

Table 2-5: Schedule of tested connection (Mourad, 1994)

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Column section (mm)</th>
<th>Beam size</th>
<th>Endplate dimension (mm)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>S5</td>
<td>254×254×11.13</td>
<td>W360×33</td>
<td>590×230×22</td>
<td>unstiffened column flange</td>
</tr>
<tr>
<td>S7</td>
<td>254×254×11.13</td>
<td>W360×33</td>
<td>590×230×22</td>
<td>concrete filled column</td>
</tr>
<tr>
<td>C1</td>
<td>254×254×11.13</td>
<td>W360×33</td>
<td>590×230×22</td>
<td>stiffened column flange  with a 6mm doubler plate</td>
</tr>
</tbody>
</table>

Figure 2-19 shows the effectiveness of each stiffening method on the typical extended endplate connection for specimens listed in Table 2-5. Filling the hollow section column with concrete increased the stiffness of the connection dramatically (three times the stiffness of unstiffened column flange), while the connection with a doubler plate increased the connection stiffness to a lesser extent. Concrete infill provided restraints to the corners and side walls thus reducing overall column face deformation. Deformation of the side walls when the hollow section was filled with concrete was almost negligible as shown in Figure 2-20.
Welding a doubler plate to the face of hollow section does not appear to result in a full composite action between the two plates due to sliding and lack of flatness after welding (Mourad, 1994). The effective column flange thickness \( t_{\text{eff}} \) due to the contribution of the doubler plate is given by Equation 2-4:

\[
 t_{\text{eff}} = \frac{1}{3} \left( t_c^3 + t_{dp}^3 \right) \tag{2-4}
\]

where \( t_c \) = column thickness and \( t_{dp} \) = thickness of doubler plate. As demonstrated by the cube root factor in this equation, the contribution of the doubler plate to the effective thickness of the column flange is not significant as the two thicknesses do not act fully compositely.

![Figure 2-19: Moment rotation curves for specimens S5, S7 and C1 (Mourad, 1994)](image-url)
Tabuchi et al. (1994) investigated a similar type of connection to Mourad’s (1994) with Huck HSBB. However, their extended endplate connection was further reinforced around the connection area by four welded steel angles to improve out of plane stiffness of the connection as shown in Figure 2-21.

Tabuchi et al. (1994) performed full scale connection assembly test (Figure 2-22) to investigate the deformability of the connection. The details of specimen tested are listed in Table 2-6. The moment rotation curve for a 400×200×8×13 H-beam is plotted in Figure 2-23. It can be seen that the connection is close to rigid for braced frames according to Eurocode 3. Although the additional angles welded to the hollow section had very high thickness, the connection still had some flexibility, rendering it unable to achieve the stiffness required for a braced frame. This reaffirms the effective thickness findings by Mourad (1994) whereby plates welded together along the perimeter do not act as a full composite member.
Figure 2-21: Extended endplate connection reinforced with welded angles (Tabuchi et al., 1994)

Table 2-6: Specimen details for tensile test (Tabuchi et al., 1994)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Column</th>
<th>Beam</th>
<th>Thickness of angle (mm)</th>
<th>T-section flange</th>
</tr>
</thead>
<tbody>
<tr>
<td>E4J-19</td>
<td>300×300×16</td>
<td>H-400×200×8×13</td>
<td>25</td>
<td>25×250×300</td>
</tr>
</tbody>
</table>

Figure 2-22: Full scale connection assembly test specimen (Tabuchi et al., 1994)
France et al. (1999a; 1999b; 1999c) conducted intensive investigations on the behaviour of simple and moment endplate connections with the Flowdrill bolting system. The investigation undertaken by France et al. (1999b) on the extended endplate connection is of primary interest as the behaviour of the connection is the closest to a rigid connection. The other connections investigated by France et al. (1999c) were the partial depth and flush endplate connections which are categorised as pinned and semi-rigid connections respectively. The extended endplate details adopted by France et al. (1999b) are shown in Figure 2-24 and details of parametric studies conducted by France et al. (1999b) are summarised in Table 2-7.
Figure 2-24: Extended endplate details (France et al., 1999b)

Table 2-7: Schedule of Flowdrill rigid joint tests (France et al., 1999b)

<table>
<thead>
<tr>
<th>Test no.</th>
<th>Endplate type</th>
<th>Column section (mm)</th>
<th>Steel grade for columns</th>
<th>Beam size</th>
<th>Endplate thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Comparison of tube wall thickness</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>Extended</td>
<td>200×200×8</td>
<td>S275</td>
<td>356×171×67 UB</td>
<td>25</td>
</tr>
<tr>
<td>20</td>
<td>Extended</td>
<td>200×200×10</td>
<td>S275</td>
<td>356×171×67 UB</td>
<td>25</td>
</tr>
<tr>
<td>21</td>
<td>Extended</td>
<td>200×200×12.5</td>
<td>S275</td>
<td>356×171×67 UB</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Comparison of column steel grade</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>23</td>
<td>Extended</td>
<td>200×200×10</td>
<td>S355</td>
<td>356×171×67 UB</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Comparison of concrete infilled (a repeat of test no. 23 with concrete infill in column)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>Extended</td>
<td>200×200×10</td>
<td>S355</td>
<td>356×171×67 UB</td>
<td>25</td>
</tr>
</tbody>
</table>
Figure 2-25 to Figure 2-27 illustrate the findings of France et al. (1999b) by comparing the effects of column thicknesses, steel grades and concrete infill on the stiffness of the connections. Stiffness classification boundaries for an assumed beam length of 7.5m based on Eurocode 3 part 1-8 (2005) are plotted on those figures.

The effects of increasing column thickness can be seen from Figure 2-25. Both the strength and stiffness of the connection increased with increasing column thickness. All connections had initial stiffnesses that exceeded the rigid classification for a braced frame. Test 19 (b/t ratio = 24) behaved as a rigid connection in a braced frame below moments of 75kNm while Test 21 with the thickest column walls (b/t ratio = 15) maintained its rigidity for a braced frame up to a moment of 200kNm. This shows a large improvement in connection stiffnesses as b/t ratio decreases due to a stiffer column flange.

![Graph showing moment rotation curves for extended endplates with varying SHS wall thicknesses.](image)

**Figure 2-25:** Moment rotation curves for extended endplates with varying SHS wall thicknesses (France et al., 1999b)
The effect of material strength is shown in Figure 2-26. At low levels of moments, both specimens behaved similarly and had the same initial stiffness. As the connection with grade S275 column had lower yield strength, yielding occurred earlier. The connection became flexible and approached its ultimate strength at lower load levels while the specimen with higher steel grade showed increasing stiffness and strength until yielding occurred at higher level of loads. Specimen with steel grade 355 (Test 23) behaved as a rigid connection for a braced frame until moments in the connection reached 155kNm while Test 20 was rigid for moments below 125kNm. This shows an increase of 24% in connection moments at the point where the connection ceased to be rigid as a result of 30% increment in steel grade.

![Moment rotation curves for comparison of SHS steel grades (France et al., 1999b)](image)

Figure 2-26: Moment rotation curves for comparison of SHS steel grades (France et al., 1999b)

Apart from investigating ways to improve the behaviour of the hollow section column by thickening the column walls and increasing steel strength, France et al. (1999b) also studied the behaviour of concrete filled joints. A dramatic increase in strength and stiffness of the concrete filled specimen compared to the unfilled
specimen is observed in Figure 2-27. Test 24 (concrete filled specimen) was able to achieve sufficient stiffness to behave as a rigid connection in a sway frame below moments of 150kNm, while for the same level of moment, its unfilled equivalent (Test 23) had reached its limit as a rigid connection for a braced frame.

![Figure 2-27: Moment rotation curves for comparison of concrete filled effects (France et al., 1999a)](image)

From the studies of France et al. (1999b), it can be concluded that concrete filling the hollow section is very effective in increasing the connection strength and stiffness, followed by increasing the tube wall thickness and finally the steel grade.

Results from the investigations by France et al. (1999b), Mourad (1994), Tabuchi et al. (1994) and Tanaka et al. (1995) indicated that it might be possible to achieve rigidity in a stiffened extended endplate blind bolted connection. Although concrete infill has proven to be very efficient in stiffening the connection, this is not considered to be practical in a low rise construction due to small column sizes and limiting the number of trades on site.
Although the extended endplate connection is commonly used for connecting beams to hollow section columns, the connection requires more fabrication work compared to a T-stub connection in terms of welding the beams to the extended endplate. The connection also poses handling issues on site and the need for tight tolerances to fit all the components together on site. In Australia, the preference is for shop welding and on-site bolting.

More recent tests on face connection to unfilled hollow section columns were conducted by Barnett (2001) and Elghazouli et al. (2009) with the Hollobolt blind bolts. Barnett (2001) tested the tension region of a T-stub connection using the same size of tubes that were used by France et al. (1999b), i.e. 200×200×8 SHS, 200×200×10 SHS and 200×200×12.5 SHS. Very thick T-stubs were used as shown in Figure 2-28 to eliminate the influence of the T-stubs. Barnett (2001) found increasing the thickness of the tubes improved the stiffness of the connection, which is expected, and also the Reverse Mechanism Hollobolt (RMH) performs better than the ordinary Hollobolt especially for the lower thickness tube due to stiffening effects provided by the RMH as the SHS face deformed.

Elghazouli et al. (2009) investigated different connections to the unfilled hollow section columns with angles. These are shown in Figure 2-29 with 150×150×6.3 and 150×150×10SHS columns connected to UB305×102×25 and UB305×165×40 beams. Three different configurations were tested; type A with only top and seat angles, type B with addition of angle cleats to upper part of the beam and type C with addition of full depth angle cleats. From the experiments it was found that column thickness has direct influence on stiffness and ultimate capacity of the connection. Further, a higher grade Hollobolt, 10.9 instead of 8.8 improves the performance of the connection when the angle has high stiffness and strength. Configuration type C is the most effective with a high initial stiffness and capacity compared to configuration type A, while configuration type B with angle cleats on
the upper part only led to a doubling of the yield moment but not much enhancement in stiffness. Although configuration type C has the highest initial stiffness, it is still well within the semi-rigid region and is suitable for braced frames or as moment resisting connections in secondary lateral resisting systems.

Figure 2-28: Deformation of T-stub connection to SHS tube (Barnett, 2001)

Figure 2-29: Connection configurations by Elghazouli et al. (2009)
It is clear from the research findings reported herein, there is a need to enhance the stiffness of connections to unfilled hollow sections in order to achieve rigid status.

2.5.2 Side shear connection

The bulk of the discussions pertaining to the shortcomings of connecting beams to the face of hollow section columns are related to the inherent flexibility of this hollow section wall. The term “side shear connection” in this thesis refers to connections that transfer forces from beams to hollow section columns via shear action on the column side walls.

Early investigations on framing connections between I-beams and SHSs have been carried out by White & Fang (1966). Five different types of simple connections between I-beams and SHSs were tested. White & Fang (1966) found that large deformation of the column face would reduce the load carrying capacity of the hollow section column significantly. A suggestion was made to concentrate welding near the corners of the hollow section to avoid excessive deformation.

Building on the concept of transferring loads from the beams to the corners of hollow section columns to prevent excessive column face deformation, Giroux and Picard (1976; 1977) studied connecting I-beams to the sides of a SHS column via coped strap angles as shown in Figure 2-30.

Details of specimens tested by Giroux and Picard (1976; 1977) are provided in Table 2-8. A comparison was made between the fully rigid butt welded connections between I-beams and H columns and strap angle connections as shown in Figure 2-31. It can be seen that the strap angle connection was not as stiff as the fully rigid connection but the connection was able to carry the full
plastic moment of the connecting beams and had adequate rotation capacity. Both the single and double sided strap angle connections behaved similarly.

![Image](plan-view-top-strap-angles.png)

**Figure 2-30: Moment connection with double sided coped strap angles (Giroux & Picard, 1976)**

<table>
<thead>
<tr>
<th>Category</th>
<th>Column size</th>
<th>Beam size</th>
<th>Strap angles</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>203×203×9 SHS</td>
<td>W8×35</td>
<td>102×102×19</td>
<td>double-sided strap angle connection</td>
</tr>
<tr>
<td>F</td>
<td>203×203×9 SHS</td>
<td>W8×35</td>
<td>102×102×19</td>
<td>single-sided strap angle connection</td>
</tr>
<tr>
<td>W</td>
<td>W8×35</td>
<td>W8×35</td>
<td>Nil</td>
<td>fully welded rigid I-beam to H-column connection</td>
</tr>
</tbody>
</table>

From the plotted stiffness gradients, it is found that the fully rigid category W connection was very stiff even for a sway frame while the strap angle connection behaved as a rigid connection in a sway frame for moments up to 0.5\(M_p\). The strap angles connection, while having successfully prevented the flexible deformation of the column face by connecting to the sides of SHS, has its limitations. The connection is only suitable for beams of the same size or smaller.
width to the SHS dimension. It is also limited to beams that are in one plane only and of the same depth in order for the bottom strap angles to hold the beams together. Also, protrusion from the strap angles may hinder the connection of orthogonal secondary beams to the primary beams near the joint region.

Building on the pioneering works by Giroux & Picard (1976; 1977), Shanmugam (1997) developed moment connections using external stiffeners based on similar concepts of connecting to the sides of the SHS. The concept of stiffening the hollow section column externally provided an alternative to the traditional method of welding an internal diaphragm plate inside the hollow section which is a complex and expensive operation. Extensive investigations were carried out by

![Figure 2-31: Moment rotation curves (Giroux & Picard, 1976)](image)

Note: W1 and W2 refer to specimens number 1 and 2 for category W, similarly for F1 and F2
Shanmugam et al. since the early 1990s to determine the most effective type of external stiffeners.

A typical specimen with external T-stiffeners is shown in Figure 2-32. Various tests were performed by Shanmugam et al. (1991; 1993) and Lee et al. (1993) on connections with different types of stiffeners to obtain the moment rotation characteristics of those connections. The behaviour of connections with T and angle stiffeners was compared to that of internal stiffeners to gauge their effectiveness. Details of some of the specimens tested are listed in Table 2-9.

![Figure 2-32: T-stiffener connection (Shanmugam, 1997)](image)

**Table 2-9: Details of specimens (Shanmugam, 1997)**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Beam size mm×mm×kg/m</th>
<th>Column size mm×mm×mm</th>
<th>Stiffener type</th>
</tr>
</thead>
<tbody>
<tr>
<td>MT1</td>
<td>305×165×38.69</td>
<td>250×250×9</td>
<td>T-stiffener (300mm long)</td>
</tr>
<tr>
<td>MT2</td>
<td>305×165×38.69</td>
<td>250×250×9</td>
<td>No stiffener</td>
</tr>
<tr>
<td>MT3</td>
<td>305×165×38.69</td>
<td>250×250×9</td>
<td>Internal stiffener (12mm thick)</td>
</tr>
<tr>
<td>MT4</td>
<td>305×165×38.69</td>
<td>250×250×9</td>
<td>T-stiffener (200mm long)</td>
</tr>
<tr>
<td>MT5</td>
<td>305×165×38.69</td>
<td>250×250×9</td>
<td>Angle-stiffener (300mm long)</td>
</tr>
</tbody>
</table>
Figure 2-33 plots the moment rotation curves for five test specimens. It can be seen that the connection with 300mm long T-stiffener (MT1) was the most stiff in comparison to other types of stiffeners. The connection with a 300mm long T-stiffener (MT1) was the most rigid; its behaviour was far better than that of the connection with the angle stiffener and performed marginally better than the connection with internal continuity plates (MT3). This shows the effectiveness of transferring loads from the beams to the sides of the SHS walls, thus eliminating the flexible column face deformation.

![Figure 2-33: Moment-rotation relationship for MT1-MT5 (Shanmugam, 1997)](image)

Similar to the connection proposed by Giroux & Picard (1976), the T-stiffener connection is only suitable for beams of the same size or smaller width to the SHS dimension. This limitation does not pose a problem in frames where strong column weak beam philosophy is adopted for earthquake design to force plastic hinges to form in the beams first. However in low rise structures especially in low...
seismicity regions, the columns are mainly designed for gravity loading resulting in frames in which the column width is often smaller than that of the beams. From a gravity design point of view, the beams, being horizontal structural elements, will carry the same load as any same span beams in a high rise structure with similar load area and floor function (i.e. retail, residential). However, the columns in a low rise structure will be subjected to much smaller loads compared to the columns in high rise structures for the same load area; hence resulting in a much smaller column size required for gravity loading. Hence the beam flanges may be wider than the SHS width rendering the T-stiffener type connection shown in Figure 2-32 unachievable.

In recent years Kumar & Rao (2006) developed a RHS beam to column connection. The RHS beam was bolted to a channel and the channel was welded to the sides of the RHS column as shown in Figure 2-34. As blind bolts were not used in this connection, a web opening was introduced to provide access to tighten the nut of a conventional bolt inside the RHS beam. This highlights the convenience of using blind bolts. It should be noted that the RHS beam can be replaced by an I-beam for the same connection configuration. The channel connectors transfer loads from the beam flanges to the column webs directly. The connection is totally independent of the column face thus avoiding the flexible column face deformation.
Kumar & Rao (2006) found that the stiffness of the connection was greatly influenced by the depth of the channel connector. Figure 2-35 and Figure 2-36 plot the moment-rotation curves for two different channel depths. It is evident that the connection with a deeper channel was much stiffer than the connection with a shallower channel, and was very rigid even for the sway frame. Although this is so, the connection with the shallower channel also had substantial stiffness for both the braced and sway frames. Again this stressed the effectiveness of utilising shear transfer to the side walls in beam-to-hollow section column connections. However, one disadvantage of this type of connection is that protrusion from the channel connectors may hinder transverse beam connections near the joint region. Also, the construction sequence of the proposed connection would be complicated if the welding of the channel connectors to the RHS column is done off site.
Even though connecting to the sides of the hollow section has the added advantage of increasing the strength and stiffness of the connection, there are no standard connections developed or in use in current practice. To date, there has been no research reported on blind bolted connections attached to the sides of hollow section column. This project will explore the feasibility of connecting to the sides of hollow section with blind bolts as an alternative to the extended endplate connection to achieve the rigidity required.
2.5.3 Summary

An extensive literature review has been presented in this chapter. The main findings of the chapter are as follows:

- Various blind bolting systems that are currently available in the market have been studied. The Ajax ONESIDE blind bolt has the advantage of being able to achieve the full structural strength of a standard bolt.

- There are a number of classification systems available to classify connections into their respective categories of stiffness, namely: pinned, semi-rigid and rigid. The Eurocode 3 classification system is one of the most popular and has been found to be relatively easy to use and has been shown to be conservative by several researchers. The Eurocode 3 classification system is adopted as the standard method of classification in this thesis.

- Current design guidelines for both the blind bolted pinned and moment connections have been reviewed. The application of these known design techniques to ONESIDE blind bolted connections has been assessed. The ultimate limit states design for a typical extended blind bolted connection is well covered and the present available design models can be directly adopted for the ONESIDE face connection with some minor modifications and verifications. However, there is a lack of information available at present on determination of stiffness for blind bolted connections.

- The literature review also covered previous studies of blind bolted connections by other researchers and highlights techniques devised to improve their performance. A typical face connection can be stiffened with a doubler plate or more effectively by increasing the column wall thickness. It is clear from the research findings reported herein that there is a need to
enhance the stiffness of face connections to unfilled hollow sections in order to achieve rigid status.

• The majority of the discussions pertaining to the shortcomings of connecting beams to hollow section columns focuses on the inherent flexibility of the column face. A method to achieve a higher connection stiffness is by transferring loads from the beams to the corners of hollow section columns to avoid flexible column face deformation, referred to herein as the side connection. Current available side connections are achieved by means of welding the connection plates to the corners of hollow section columns. While such connections are structurally efficient, the required fabrication is not necessary cost effective.

The major areas which require further research and are considered in this thesis are summarised as follows:

▪ Maximise connection stiffness utilising on-site bolting with minimal prior welding and fabrication. Such connections can be possibly achieved by minimising or eliminating flexibilities associated with column face deformations.

▪ Develop design models for determining the stiffness of the blind bolted connections developed, hence providing guidance for designers as to the stiffness values to adopt in their design
Chapter 3
Preliminary Work

3.1 Introduction

This chapter presents the initial stage of investigating the deformability of the hollow section column face subjected to bolt pull-out as the inherent flexibility of the column face is a critical parameter determining the stiffness and strength of a typical face connection. The load-deformation behaviour of a hollow section subjected to a pull-out force from a bolt is influenced by:

1) Width to thickness (b/t) ratio of hollow section
2) Endplate thickness
3) Bolt pretension
4) Location of bolts on column face
5) Size of bolt and bolt hole

The influence of each parameter on the deformability of the column face was gauged using pilot finite element (FE) analysis. The findings from this work form the basis for the experimental study and advanced FE modelling in subsequent chapters.
3.2 Study of bolt pull-out behaviour

Ivanyi (2008) investigated bolt pull-out behaviour of various thicknesses of steel plate in a bolted steel connection. Experimental setup and specimen details are shown in Figure 3-1. The plate was fixed to a rigid frame by M6 bolts and a bolt is placed in the centre hole. The load was applied increasingly to the bolt until the bolt was pulled-out of the plate. Figure 3-2 shows the failure mode for 1 mm thick steel plate.

![Test specimen by Ivanyi (2008)](image)

To gain an understanding of bolt pull-out behaviour on the hollow section tube and assess the feasibility of reproducing experimental results using a commercial FE software, a simple FE model was created to simulate the test behaviour from Ivanyi (2008). A quarter symmetry model of the test specimen was created using ANSYS as shown in Figure 3-3. The edges of the plate which is located on the centreline of the fixing bolts is restricted in all directions. The material properties of the plate adopted in the model are as suggested by Ivanyi (2008): elastic
modulus, \( E = 210000 \text{MPa} \), Poisson’s ratio of 0.292, yield stress of 210MPa and tangent modulus of 1010MPa. Load is applied as a uniform pressure around the circumference area of the bolt hole to represent the pull-out action by the bolt head. The deformed shape of the plate from the FE model is shown in Figure 3-4 which is similar to the failure mode shown in Figure 3-2

![Figure 3-2: Failure mode for 1 mm plate thickness (Ivanyi, 2008)](image)

Comparison of the test results by Ivanyi (2008) and the simple FE model shows good agreement (refer to Figure 3-5) giving confidence that the FE model, despite its simplicity, is able to simulate deformation of steel plate under applied load. This led on to the next section, extending the simple model to a hollow section tube, and investigating parameters that will affect the deformability of the flexible tube face.
Blind Bolted Connections for Steel Hollow Section Columns

Figure 3-3: Finite element model (quarter symmetry)

Figure 3-4: Deformed shape
3.3 Preliminary tube model

In this section, simplified FE models employing eight-node solid elements (SOLID 45) of a 150×6 SHS with four M16 bolt holes are analysed. Each bolt hole is subjected to a uniform pressure equivalent to 10kN to simulate upward pressure on the washer area due to bolts acting in tension as shown in Figure 3-6. The models analysed are listed in Table 3-1 and they are assigned linear elastic material properties for simplicity as summarised in Table 3-2. The purpose of these models is to assess the deformability of the hollow section and how it is affected by stiffening the column flange and side walls assuming a full composite action between the hollow section and stiffening plates. Case A considers the hollow section by itself without any stiffening, while in case B, a 6mm plate is fully glued onto the column face, and in case C 6mm side plates are glued onto the column side walls over the connection area.
Table 3-1: List of simplified tube models

<table>
<thead>
<tr>
<th>Case</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>hollow section by itself</td>
</tr>
<tr>
<td>B</td>
<td>Stiffened with 6mm top plate</td>
</tr>
<tr>
<td>C</td>
<td>Stiffened with 6mm side plates</td>
</tr>
</tbody>
</table>

Table 3-2: Material properties for model

<table>
<thead>
<tr>
<th>Element</th>
<th>Young’s modulus, E (MPa)</th>
<th>Yield Stress, $\sigma_y$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hollow Section</td>
<td>200000</td>
<td>350</td>
</tr>
<tr>
<td>Plate</td>
<td>200000</td>
<td>300</td>
</tr>
</tbody>
</table>

Figure 3-6: Simplified tube model, case A
The deformed shapes and principal stresses for the FE models conducted are shown in Figure 3-9 to Figure 3-11. The maximum deformation and stresses at the column face and corners for each model are recorded in Table 3-3.
Table 3-3: Summary of results for simplified tube models

<table>
<thead>
<tr>
<th></th>
<th>Max deflection at column face, $\Delta$ (mm)</th>
<th>Stress @ mid face (MPa)</th>
<th>Stress @ corner (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hollow tube - original</td>
<td>0.91</td>
<td>270</td>
<td>350</td>
</tr>
<tr>
<td>Flange strengthen</td>
<td>0.27</td>
<td>120</td>
<td>120</td>
</tr>
<tr>
<td>Web strengthen</td>
<td>0.64</td>
<td>210</td>
<td>400</td>
</tr>
</tbody>
</table>

It can be observed that stiffening the column flange is the most efficient with dramatic reduction in column face deformation and stresses. However, these models assume a full composite action between the plates and the tube walls. In reality, this can be difficult to achieve. The next section of this chapter investigates the effectiveness of bolt pretension in clamping two plates together.

Figure 3-9: Deformed shape of Case A, hollow section by itself
Figure 3-10: Maximum column face and corner stresses (Case A)

Figure 3-11: Deformation at middle cross section for cases B and C
3.4 Bolt pretension

Section 2.5.1 of the thesis discussed methods to improve the stiffness of the endplate connection; one such method is by welding a doubler plate on the column face. Equation 2-4 proposed by Mourad (1994) suggests that the increase in effective thickness obtained by welding two plates together along the perimeter is minimal. In this section, simple FE models are developed to assess the effectiveness of bolt pretension in stiffening and improving the effective thickness of two plates that are clamped together by bolting.

A simple FE model with two square plates and a bolt as shown in Figure 3-12 is modelled as part of the sensitivity analysis to determine the effects of bolt pretension. The plates are 60×60×6 mm with an M16 Ajax ONESIDE bolt with bolt yield stress of 660MPa at the centre. A total of 80kN is applied around the perimeter of the top plate while the bottom plate is rigidly fixed around the perimeter.

To simulate pretension effects in the bolt, a pretension section is created in the bolt shank with PRETS179 elements and 70kN of pretension load (~70% of ultimate bolt capacity) is applied to this section. The PRETS179 element is used to define a 2-D or 3-D pretension section within a meshed structure. The PRETS179 element has one translational degree of freedom and a pretension node, which provides a convenient way to assign boundary conditions on an entire pretension section (ANSYS Inc., 2007). The specified pretension load is applied incrementally to the model during the first load step. The initial displacement at the pretension section is locked at the end of the first load step. Once the bolt is fully pretensioned and the initial displacement is locked, external load is applied incrementally in the next load step. The PRETS179 pretension element allows direct input of preload force which simplifies the modelling of the pretensioned...
This approach of modelling pretension in bolts has been adopted previously by Yao (2009) and has been found to be an effective and reliable method.

Various cases are investigated to identify the conditions when bolt pretension is most effective. The first case (A1 and A2) is an idealised scenario whereby all components except for the bolt shank are very rigid with a Young’s modulus, $E = 200,000$ GPa which is 1000 times the normal $E$ for steel. The second case (B1 and B2) is when the top plate and bolt shank are flexible (normal $E$ for steel = 200 GPa), all other components remain rigid and the last case (C1 and C2) is a more realistic one with all components having a normal $E$ of 200 GPa. For simplicity, linear elastic material properties are assigned to all models. The “all deformable” model with normal steel modulus (i.e. cases C1 & C2) is also assigned bilinear elasto-plastic material properties with a yield stress of 350MPa and tangent modulus of 10% $E$ for comparison purposes. A list of the FE analyses conducted to investigate the effect of pretension is given in Table 3-4.
Table 3-4: Parametric analysis for bolt pretension

<table>
<thead>
<tr>
<th>Cases</th>
<th>RIGID $E=200,000,\text{GPa}$</th>
<th>DEFORMABLE $E=200,\text{GPa}$</th>
<th>Material Behaviour</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1 – without pretension</td>
<td>plates, washers, bolt head, nut</td>
<td>bolt shank</td>
<td>elastic</td>
</tr>
<tr>
<td>A2 – 70kN pretension</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B1 – without pretension</td>
<td>plates, washers, bolt head, nut</td>
<td>top plate, bolt shank</td>
<td>elastic</td>
</tr>
<tr>
<td>B2 – 70kN pretension</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C1 – without pretension</td>
<td>nil</td>
<td>all deformable</td>
<td>elastic and bilinear</td>
</tr>
<tr>
<td>C2 – 70kN pretension</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 3-13 shows the contact pressure between top and bottom plates for the “all deformable” model (Case C2) with elasto-plastic material properties. It is observed that the contact area exerted by bolt pretension spreads out to approximately double the washer area with most contact pressure concentrated around the bolt hole and decreasing with distance away from the bolt hole. As the clamped area produced from bolt pretension is only confined to a small area around the washer and contact pressure falls rapidly away from the bolt hole, it is expected that the pretension in the bolt will not result in a significant effective thickness for plates that are clamped together by bolt pretension. The ONESIDE sleeve was not modelled. This should not affect the clamping pressure as the sleeve is shorter than the gap within the washers, allowing the plates to compress freely under the pretension load.
Results from the FE models for four scenarios: (i) both plates rigid (Cases A1 & A2), (ii) top plate flexible (Cases B1 & B2), (iii) all components deformable (Cases C1 & C2) – elastic material properties and (iv) all components deformable (Cases C1 & C2) – bilinear material properties are plotted in Figure 3-14 and Figure 3-15. Tensile stress of the bolt, $\sigma_1$ is plotted in Figure 3-15 as the bolt is acting predominantly in tension when the load is applied.

Both figures indicate that only when both plates are rigid (scenario (i)), the full pretension effect can be observed. The plate remains clamped and tensile stresses in the bolt remains constant up to an applied force of 70kN (which is the pretension load). When the applied force is equivalent to the pretension load in the bolt, the plates start to separate and stresses in the bolt start to increase indicating that pretension in the bolt has been overcome. For scenario (ii) when
the top plate is assigned the normal $E$ value, an early release of pretension load is observed, with increasing plate separation and stresses in the bolt when the applied load is close to 20kN. For the final two scenarios when all components in the model are deformable, the pretension load is relieved at an early stage, with the bilinear model being much more flexible than the linear elastic model due to yielding of the plates. Overall, it is observed that bolt pretension has a stiffening effect and gives a lower deformation compared to when there is no pretension. However, the stiffening effect from bolt pretension is most effective when the plates are very rigid.

(i) rigid model (A1 vs. A2)
(ii) top plate flexible (B1 vs. B2)

(iii) all flexible (C1 vs. C2) – elastic material properties
Blind Bolted Connections for Steel Hollow Section Columns

(iv) all flexible (C1 vs. C2) – bilinear material properties

Figure 3-14: Load-displacement graphs for bolt shank

(i) rigid model (A1 vs. A2)
(ii) top plate flexible (B1 vs. B2)

(iii) all flexible (C1 vs. C2) – elastic material properties
3.5 Summary

Several preliminary FE studies have been reported in this chapter. They have been undertaken to enable confidence to be developed in the modelling techniques and to examine the key fundamental behaviour that is associated with bolt pull-out, tube front face flexibility and bolt pretension. The preliminary FE work conducted in this chapter is able to simulate experimental results of bolt pullout behaviour from a steel plate which forms the basis of flexibility in the steel tube when subjected to tension action from the connecting bolts.

Confidence was gained that the simple FE model is able to predict behaviour of steel plate deformation subjected to bolt pull-out, and the model was then extended to a hollow section tube. The behaviour of hollow section tube with and without stiffening and the influence of bolt pretension were explored. It was found that the most effective way of improving stiffness of the tube subjected to bolts pull-out was by increasing the column face thickness. Preloading the bolts
also helped to increase stiffness of the tube face. Effective clamping area of the pretension bolt is concentrated around the washer area. Clamping pressure is maximum inside the washer area and reduced significantly to zero at approximately twice the washer diameter.

In subsequent chapters, various configurations for blind bolted connections to hollow section tube are explored and tested. The FE model developed in this chapter is further extended to model more complex connections, simulate experimental behaviour and perform sensitivity analysis to optimise the various connections investigated.
Chapter 4

Blind Bolted T-stub Connection

4.1 Introduction

The blind bolted T-stub connection is a simple face connection similar to the extended endplate connection except the beam flanges are bolted to the T-stem instead of the cross section of the beam being welded to the endplate as in case of the extended endplate connection. By eliminating the need for welding of beam to the endplate, the T-stub connection has a marked advantage in the low rise construction industry, since fabrication work is minimised to reduce cost and construction time. The T-stub itself can be cut from a standard universal or welded beam (UB or WB). The T-stub connection is assessed as an alternative connection to the welded connection currently used as a moment connection for unfilled hollow section columns in the Australian construction industry (refer to Figure 1-2).

In this chapter a T-stub connection is investigated for smaller columns than the ones previously tested by other researchers. These smaller size columns are typical
of those used in low rise residential and commercial structures. The connection relies upon commercially available Ajax ONESIDE blind bolts, a different type of blind bolt connector to those investigated previously. The chapter aims to investigate the behaviour of the connection and explore parameters that affect its stiffness. Design models for strength and stiffness of the T-stub connection are also developed. Based on the experimental results, the behaviour of the connection in terms of stiffness is assessed in accordance with the EUROCODE 3 classifications and the strength of the connection is compared with established design models.

4.2 Design concept

In accordance with the International Institute of Welding (IIW) (1989), a T-stub connection should be designed to transfer a limited amount of tension force in order to ensure that the column face deformation does not exceed 1% of the column width at the serviceability limit state and does not exceed 3% of the column width at the ultimate limit state. These deformation limits are likely to govern the maximum design force in the T-stubs rather than strength considerations. The experimental study presented in Section 4.3 has been carried out to determine the actual ultimate capacity as well as the stiffness of the T-stub connection.

In areas of low to moderate seismicity such as Australia, the columns tend to be small in size compared to the beams as the frames tend to be governed by gravity loading. Hence, the strength hierarchy at the joint is such that the column is expected to reach its capacity before the connection or the beam. An initial upper bound estimate of the tension force transferred by the T-stub can be made by dividing the column section bending capacity by the beam depth. For example, for a 150x6 SHS column and a 300 mm deep beam (sizes commonly used in the low rise construction industry) this would correspond to an ultimate tension force
of 170kN to be imposed on the T-stub. This initial estimate of ultimate tension force is used later when estimating the range of loadings that is relevant in a practical situation and is compared with the connection behaviour in Section 4.4.

4.3 Experimental program

4.3.1 Test specimens

In total three T-stub specimens were tested and the results of these tests are described herein. The test specimens were constructed to simulate the tension (S1 and S2) and compression (S3) regions of a proposed T-stub connection as shown in Figure 4-1. Details of the specimens are given in Table 4-1 and Figure 4-2. The T-stubs used in this chapter were fabricated by welding plates together, due to small amount required for testing.

Figure 4-1: Proposed T-stub connection
Blind Bolted Connections for Steel Hollow Section Columns

Figure 4-2: Specimen detail (S1, S2, S3)

a) Elevation view of specimen

b) Plan and front views of specimen

*All dimensions in mm
Table 4-1: Details of specimens

<table>
<thead>
<tr>
<th>Specimen Number</th>
<th>Column section (mm)</th>
<th>Column length (mm)</th>
<th>Endplate thk. (mm)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1 (T)</td>
<td>150×6 SHS</td>
<td>1300</td>
<td>10</td>
<td>no bolt sleeves</td>
</tr>
<tr>
<td>S2 (T)</td>
<td>150×6 SHS</td>
<td>1300</td>
<td>10</td>
<td>with bolt sleeves</td>
</tr>
<tr>
<td>S3 (C)</td>
<td>150×6 SHS</td>
<td>1300</td>
<td>10</td>
<td>with bolt sleeves</td>
</tr>
</tbody>
</table>

**Note: (T) denotes tension test and (C) denotes compression test**

All specimens were identical except that bolt sleeves (component C in Figure 4-3) were utilised in Specimens S2 and S3 only. All specimens were comprised of unfilled SHS columns with a cross section of 150×150×6mm and endplate of 10mm thickness. The ONESIDE bolts used in the tests can develop the full structural strength of a standard Grade 8.8 M16 (16mm diameter) bolt with a minimum tensile strength of 800 MPa and yield strength of 640 MPa. The ONESIDE bolts require oversize bolt holes; for example a 24mm hole for an M16 ONESIDE bolt. The gap between the bolt and the bolt hole is reduced by the placement of a sleeve, which is also made of the same material as the bolt. The sleeves used in these experiments had tight tolerances to the bolt holes. The effect of varying this tolerance was investigated in the finite element sensitivity studies discussed in Section 4.5. The bolts in the specimens were tensioned to the snug tight condition.

![Figure 4-3: Ajax ONESIDE bolt assembly and installation tool (Ajax)](image)

In order to determine the material properties, tensile tests were also performed on six coupon “dog-bone” specimens cut from three faces of two 150×150×6mm SHS
tubes (for each tube, two specimens were cut from the faces adjacent to the weld seam while one specimen was cut from the face opposite to the weld seam of the tube). All locally produced hollow sections in Australia are cold formed, the 150×150×6mm SHS is designated as C350 i.e. cold formed section with nominal yield stress of 350MPa. The average yield stress for the six specimens is 424MPa with a standard deviation of 15MPa. The face opposite the weld seam has a slightly higher yield stress compared to the faces adjacent to the weld seam. The stress strain curves for coupons taken from a 150×150×6mm SHS is shown in Figure 4-4.

![Stress-strain curves from coupon tests](image)

**Figure 4-4: Stress-strain curves from coupon tests (specimens A1, A2 and A3)**

### 4.3.2 Experimental setup and instrumentation

The test setup is shown in Figure 4-5 and Figure 4-6. Loading was applied to the test specimens via the stem of the T-stub by gradually increasing the load up to failure. The specimens are held down at the ends by channel sections bolted to
strong floor below. An illustration of the instrumentation used to monitor the behaviour of specimen S2 is given in Figure 4-6. Both specimens S1 and S3 were instrumented in a similar manner to specimen S2. Linear Variable Displacement Transducers (LVDTs) were mounted on the specimen to measure the deformation of the endplate and tube wall relative to the ground. Strain gauges were placed at various locations to monitor the strains in the endplate and tube walls. A special type of strain gauge (shown in Figure 4-6) was embedded inside two of the bolts in Specimen S2 only. These strain gauges (bolt gauge) were used to measure strains in the bolt shank.

Digital photogrammetry, a three-dimensional coordinate measuring technique, was utilised to provide an overall deformation profile of each test specimen. This in-house developed photogrammetry system has an accuracy of approximately 0.03mm. The photogrammetry measurements were taken at different stages of testing with the first survey recording the original coordinates of the targets just prior to loading. Subsequent surveys were taken when the load was applied to the T-stub to record the new coordinates of the targets at each load level. By analysing consecutive overlapping photographs at each load interval, the topographic information for the specimen can be established. An initial survey was
taken prior to each test and subsequent surveys were taken at load intervals of 25kN up to 250kN for tension specimens S1 and S2, and up to 350kN for compression specimen S3. Thereafter the specimens were loaded continuously to failure.

4.4 Experimental results

4.4.1 T-stub strength and failure modes

4.4.1.1 Tension region

At low levels of applied load the endplate started to deform upwards at the intersection with the T-stem, and the endplate separated from the tube face. As the load was increased, a larger separation was observed; the deformed profile of the endplate became a double curvature profile due to the clamping action
provided by the snug-tight bolts as shown in Figure 4-7. The bolts tilted away from the T-stem indicating that the bolts were subjected to lateral bending and shear as well as tension. There were significant localised bulging effects on the tube face around the bolt holes due to bearing action of the bolt heads and internal washers.

![Figure 4-7: Deformed shape of specimen S2 at high level of applied load](image)

The blind bolts in Specimen S1 did not have sleeves. The gap between the bolt shank and bolt hole was 8mm total (16mm diameter bolt, 24mm diameter bolt hole). Specimen S1 (without bolt sleeves) failed at a load of 280kN, with bolts at one end of the stem failing in a brittle manner under the combined action of tension and bending. The fractured bolts, as well as a severely deformed one from the other end of the stem, are shown in Figure 4-8. Figure 4-8 also shows the deformed shape of the tube and endplate after failure.
For specimen S2, the presence of the sleeves which fill the gap between the oversized hole and bolt shank (reducing the gap from a total of 8mm to 0.5mm) reduced the bending of the bolts. This in turn increased the ultimate strength. The stiffening action provided by the bolt sleeves reduced the likelihood of bolt failure. In fact, bolt failure was preceded by a different failure mode in which the tube face around the bolt hole area tore open, leading to some bolts pulling out from the tube face (refer Figure 4-9 and Figure 4-10). This punching failure occurred at a load of 300kN. Both failure loads for specimens S1 and S2 were much higher than the ultimate design load for the connection as discussed in Section 4.2.
4.4.1.2 Compression region

As expected, specimen S3, which was loaded in compression, experienced much less deformation than that experienced by specimens S1 and S2 in tension. Failure occurred when the side walls directly underneath the endplate began to crush.
under the applied compressive load (refer Figure 4-11) as the side walls bulged outwards (refer Figure 4-12). The endplate appeared to form hinges across the intersection of the stem and deformed in a bow shape. The maximum load sustained by the specimen was approximately 400kN.

![Figure 4-11: Specimen S3 at failure](image)

![Figure 4-12: Crushing of side walls directly underneath the endplate (specimen S3)](image)
4.4.2 T-stub stiffness

The overall deflection of the stem is the aggregate deformation of the endplate, bolts and tube. For tension specimens S1 and S2, the endplate initially exhibited little deformation. The endplate deformed in a double curvature profile as the applied load increased. Yielding in the endplate occurred across the bolt lines and at the intersection with the T-stem. Both specimens S1 and S2 had a similar value of initial stiffness, although specimen S2 (with bolt sleeves) had a greater stiffness at loads exceeding 50kN (refer Figure 4-13). This can be explained in terms of the sleeves effectively restraining further slip of the endplate once it is in contact with the sleeves. The sleeves also acted as stiffeners, reducing the amount of bending in the bolts, and hence creating a stiffer connection overall. The deformation limits recommended by the International Institute of Welding (IIW) (1989) are also plotted on Figure 4-13. The serviceability limit state deformation of 1% of column width corresponds to an applied load of 50kN, while an applied load of 100kN corresponds to the at ultimate limit state deformation of 3% of column width. As mentioned in Section 4.2 deformation limits clearly govern the design of the T-stubs rather than strength.

The endplate in Specimen S3 loaded in compression experienced a linear load-deformation relationship up to an applied load of 300kN (refer Figure 4-14). The deformation of the endplate started to increase at a faster rate beyond this load. The load remained constant at 340kN while the deformation increased dramatically due to plastic deformation of the endplate and outward deformation of the tube side walls. The stiffness in compression region is much higher than specimens S1 and S2. At an applied load of 100kN (range of loading controlled by the tension side), the compression region is approximately five times stiffer than the tension region. This is discussed further in Section 4.6.2.4.
Figure 4-13: Comparison of endplate displacement between specimens S1 and S2

Figure 4-14: Endplate displacement for specimen S3
4.5 Finite element (FE) analysis

4.5.1 Contact elements and material properties

A three-dimensional detailed FE model was built using the general purpose software ANSYS to represent the blind bolted T-stub connection to an unfilled SHS column. The FE model took into account material and geometric non-linearities. As per Section 3.3, eight-node solid elements (SOLID 45) were used to model all components of the connection, i.e. SHS column, T-stub and bolts. Complex contact interactions between the following elements were also included (shown in Figure 4-16):

1) endplate and tube face,
2) external bolt washers and endplate,
3) internal bolt washers and interior tube face,

Additional contact surfaces were included for specimens S2 and S3:
4) bolt sleeve external surface and endplate around bolt hole,
5) bolt sleeve external surface and tube around bolt hole, and
6) bolt sleeve internal surface and bolt shank.

Surface to surface contact elements with friction coefficient of 0.25 were employed in the FE model. Each contact pair is made up of CONTA174 which is an 8-node surface to surface contact element and TARGE170 which is the paired target surface. The contact pair is sandwiched between the surfaces of 3D solid elements (SOLID 45). It has the same geometric characteristics as the solid elements immediately above and below the contact surface.

Taking advantage of symmetric conditions along the longitudinal and transverse planes, only a quarter of each test specimen was modelled. The finite element
model of the T-stub connection is shown in Figure 4-15 (full model shown for ease of visualisation) and the various contact surfaces are shown in Figure 4-16.

![Figure 4-15: Full FE model of specimen](image)

The material properties of the SHS column, T-stub and bolts were described by bilinear stress-strain curves based on the von Mises yield criteria with rate independent isotropic work hardening. The yield stress and tangent modulus of the various elements are summarised in Table 4-2. As part of a sensitivity analysis, the steel stress-strain relationship for the SHS column was modelled as a multilinear curve to best match the experimental coupon results. The results were compared with the bilinear assumption and were found to be in excellent agreement (Figure 4-18). Hence the simpler bilinear model of the material behaviour was adopted for the SHS column, since it gave sufficiently accurate results.
Blind Bolted Connections for Steel Hollow Section Columns

a) Cross section of bolt

b) Contact surfaces

Figure 4-16: Contact surfaces in FE model
Table 4-2: Material properties for bilinear model

<table>
<thead>
<tr>
<th>Item</th>
<th>Yield Stress (MPa)</th>
<th>Elastic modulus, $E_o$ (MPa)</th>
<th>Tangent modulus, $E_{tg}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SHS</td>
<td>350</td>
<td>200000</td>
<td>$0.01E_o = 2000$</td>
</tr>
<tr>
<td>T-stub</td>
<td>300</td>
<td>200000</td>
<td>$0.05E_o = 10000$</td>
</tr>
<tr>
<td>Bolt</td>
<td>640</td>
<td>200000</td>
<td>$0.01E_o = 2000$</td>
</tr>
</tbody>
</table>

4.5.2 Comparison of FE predictions to experimental results

4.5.2.1 Deformation

The FE model was validated with experimental results from LVDT readings and photogrammetry surveys. Figure 4-17 and Figure 4-18 show a very good match between LVDT readings and predictions from the FE model of the endplate displacement for specimens S1 and S2 respectively.

Figure 4-17: Comparison of FE model and LVDT readings for endplate deformation (specimen S1)
A similar comparison was made for specimen S3 (refer Figure 4-19). Figure 4-19 shows that the FE model is able to represent the initial stiffness of the specimen quite closely up to a load of 200kN, beyond which the FE model is more flexible than the specimen tested. At a load of 200kN the corners of the tube yielded extensively in the FE model accounting for the flexibility shown in Figure 4-19. This flexibility and lower capacity of the FE model is likely to be attributed to the higher yield stress and strength of the actual steel along the corners of the tube resulting from the additional work hardening required to cold form the corners. There is a good match between the FE model and LVDT readings within the practical range of loading likely to be experienced by the T-stub connection (refer to earlier discussion in Section 4.2).
While the experimental and FE overall load deflection curves are in good agreement, this is not sufficient to fully validate the FE model. The overall deflection of the stem is the aggregate deformation of the endplate, bolts and tube. Hence, to fully validate the FE model, the deformations of each of these components should be assessed separately. Therefore, the results from the FE model results were further validated using the detailed photogrammetry surveys.

The photogrammetry targets which were attached to the tube walls and T-stub were carefully matched with the nodes on the FE model. The displacements from the FE models were then compared with those from the photogrammetry surveys and both were found to be in good agreement. Figure 4-20 shows comparisons between FE and photogrammetry deformations of the endplate ($\Delta_Y$) at different levels of applied load. It is clear that the FE model replicates the endplate deformed profile well. The FE deformations and photogrammetry results for the tube side wall also matched well (refer Figure 4-21). These comparisons highlight the advantage of employing photogrammetry measurements to obtain the entire
3D deformation profile of the specimens as compared to the use of LVDTs to measure localised deformation in one direction only.

Figure 4-22 compares FE prediction with photogrammetry target at corner of Specimen S3. The legend “Phtgrmy-east” in the graph denotes results for photogrammetry target at east corner of the tube and similarly “Phtgrmy-west” denotes the target at west corner. It can be see that the FE model underpredicts the capacity of the tube corner as discussed earlier in Figure 4-19.
b) Endplate deformed profile for different applied loads
Figure 4-20: Comparison of results for endplate longitudinal deformed profile between FE model and photogrammetry (Specimen S1)

a) FE model indicating horizontal displacement of tube side wall
b) Tube side wall deformed profile at different applied loads

Figure 4-21: Comparison of results for side wall deformed profile between FE model and photogrammetry at different load levels (Specimen S1)

a) location where results for FE model and photogrammetry targets are compared
4.5.2.2 Bolt strains

Figure 4-23 shows a comparison between the strains predicted by FE model and those measured by bolt strain gauges in specimen S2. It was observed that the strains in both instrumented bolts were similar and, although designated to be “snug tight”, there appears to be a 50kN combined pretension in the bolts (i.e. 12.5kN in each bolt) due to tightening of the bolts by hand which was not accounted for in the FE analysis. This initial clamping was observed from the pressure film. The slope of the FE and experimental load-strain curves matched well between 50 and 200kN. Thereafter the strains recorded by the strain gauges increased significantly to beyond the yield strain. The experimental strain measurements at higher loads were not reliable as the bolts were subjected to bending in addition to tension forces.
4.5.3  Sensitivity analyses

Sensitivity analyses for specimens S1, S2, and S3 have shown very good correlation with experimental results and hence are reliable for use to perform sensitivity analyses. Three parameters are considered in the sensitivity analyses presented in this chapter. These are endplate thickness, size of bolt sleeves, and coefficient of friction ($\mu$) as discussed in the following subsections.

4.5.3.1  Varying endplate thickness

It was anticipated that increasing the endplate thickness would markedly improve the stiffness of the connection, and the first set of analyses aimed to explore the effectiveness of this strategy. Two simple analyses were undertaken: one with an endplate thickness of 20mm and the other with a very stiff endplate with Young’s modulus, $E = 200,000$ GPa (one thousand times the $E$ value of steel). The load-deformation curves for the T-stub in tension are shown in Figure 4-24. It can be seen that doubling the endplate thickness from 10mm to 20mm greatly improved...
the initial and post elastic stiffness. However, the endplate with very high stiffness only improved the stiffness of the connection slightly as compared to the model with 20mm thick endplate. This indicates that, with this configuration, little or no further benefit to the stiffness can be achieved by increasing the endplate thickness above 20 mm.

![Graph showing sensitivity analysis for endplate thicknesses](image)

**Figure 4.24**: Sensitivity analysis for endplate thicknesses

### 4.5.3.2 Tolerance between bolt hole and outer diameter of sleeve

In specimen S2 there was a very tight tolerance (close to nil) between the outer diameter of sleeves and the bolt holes. This provided a very tight fit for the connection which is not practical for most cases. Hence, a sensitivity analysis was carried out on a model with a clearance of 2mm between the outer diameter of sleeves and the bolt holes for an endplate thickness of 10mm. This 2mm tolerance is typical in practice for bolted connections with a bolt size smaller than M24. The bolt hole diameter for M16 blind bolt was 24mm while outer diameter of the sleeves modelled was 22mm. The thickness and the outer and inner diameters of
the sleeves were adjusted in the FE model to keep the hole and bolt sizes the same for both models. It was also assumed that no pretension was applied to the bolts.

From the load displacement curve in Figure 4-25, it can be seen that when a 2mm tolerance is present between the sleeves and the bolt hole, the overall behaviour is very similar to the model without sleeves. The model with sleeves of tight tolerance (specimen S2) forms the upper bound for initial stiffness while the model without bolt sleeves (specimen S1) has the lowest initial stiffness. The stiffness of the model with sleeves of 2mm tolerance falls in between the two models. In the range of interest for serviceability loading (below 60kN applied load), the stiffnesses for all three cases are very close. Once the displacement is high enough, the sleeves with 2mm tolerance bear on the tube and the endplate, hence a higher stiffness is observed for the connection with 2mm sleeve tolerance above 160kN.

![Figure 4-25: Sensitivity analysis for sleeves’ bolt hole tolerance, 10mm endplate](image)
4.5.3.3 Coefficient of friction, $\mu$

The coefficient of friction, $\mu$ between contact surfaces was varied from 0.25 in the original FE model for Specimen 2 to 0.15 to investigate the effect of friction coefficient to connection stiffness.

Figure 4-26 shows the comparison between load displacement curves for Specimen 2 using friction coefficient, $\mu$ of 0.25 and 0.15 between contact surfaces. It can be seen that $\mu$ does not affect the initial stiffness of the connection and lowering $\mu$ only has slight effect on the post elastic stiffness. This is because the connection is acting mainly in tension and $\mu$ is important when loads from the beam are transferred to the column in shear.

![Figure 4-26: Sensitivity analysis for coefficient of friction, $\mu$ between contact surfaces](image)

4.5.4 **Modeling of overall T-stub connection**

In order to assess the behaviour of the full connection, a FE model of the entire connection (including the tension and compression zones) was developed as
shown in Figure 4-27. The main purpose of this full model was to determine the initial rotational stiffness for the whole connection rather than the axial stiffness of the individual tension and compression zones. A 10mm endplate with zero tolerance has been adopted in the full model to match the specimens used in the experiment. In the model of the full connection it was assumed that there is no relative slip between the T-stub stem and the beam. In reality there may be some slip induced but this is assumed to be small in the serviceability range due to the use of fully tensioned (TF) bolts. It should be noted that the connections along the beam flanges can be made using standard bolts, and hence the slip load can be obtained using standard design criteria. These depend on the friction coefficient and number of bolts. It has been assumed here that the slip load is not exceeded.

A point load is applied at the shear centre of the beam and comparisons of displacements for the full and individual models are taken at the intersection of the endplate and T-stem (as shown in Figure 4-27) with reference to the tube (column) corner.

Figure 4-28 shows comparisons between the initial load-displacement curves for the full model and corresponding models for the tension and compression zones. Bending effects in the column were subtracted from the overall displacements in Figure 4-28. The stiffnesses from individual models in tension (specimen S2) and compression (specimen S3) match well with the stiffness from the full model. This implies that the tension and compression T-stubs act independently to each other and can be modelled and analysed separately, at least in the serviceability range of loading before bolts in the beam start to slip.

The load-displacement curve from the full FE model has been converted to a moment-rotation curve for a 310UB32 (approximately 310mm deep, 32kg/m section) beam, a size typically used in low rise construction as shown in Figure 4-29. The initial rotational stiffness, $S_{\text{ini}}$, of the connection is classified in
accordance with Eurocode 3 Part 1-8 (European Committee for Standardisation (CEN), 2005). In Eurocode 3 a connection is classified as:

1) rigid: if $S_{j,ini} \geq k_b E_{b}/L_b$ where $k_b = 8$ (braced frames) and $k_b = 25$ (sway frames);
2) nominally pinned: if $S_{j,ini} \leq 0.5 E_{b}/L_b$;
3) semi-rigid: for cases in between the rigid and nominally pinned.

The classification limits are dependent on the connected beam properties where $E_{b}$ is the flexural stiffness of the beam and $L_b$ is the span of the beam. Hence, assuming a 6m beam length as is common for low rise residential structures, the classification limits based on the properties of 310UB32 beam are included on Figure 4-29. It can be seen that the stiffness of the T-stub connection is well within the semi-rigid region. Even though the connection is not considered a rigid connection, its nominal stiffness can be very useful as part of the lateral load bracing system for resisting part of the wind loads. In order to assess the stiffness and strength of the connection, a simple design model is discussed in the following section.
Figure 4-27: FE model of full beam-to-column T-stub connection (elevation view)

- **a)** Load is applied at shear centre of UB, 1m away from the face of endplate.
- **b)** Displacements are taken at center of endplate-stem intersection for both tension and compression regions.

Load

**Δc** measured at this point with reference to tube corner

**Δt** measured at this point with reference to tube corner

- **K** specimen S2 = 37 kN/mm
- **K** full model = 40 kN/mm

**Δt** – displacement in tension region

**Δc** – displacement in compression region

a) Comparison of stiffness in tension region between FE model for specimen S2 and full connection.
b) Comparison of stiffness in compression region

Figure 4-28: Comparison of stiffness between full and partial FE model

Figure 4-29: Moment rotation curve for 150x6 SHS, full T-stub connection.

**Note: Slip from beam has not been accounted for in the graph and displacement is taken with reference to column corner**
4.6 Design model

In this section simplified design models are formulated for the T-stub connection to predict the strength and stiffness. A summary for the overall design of the T-stub connection is given in Figure 4-30.

In terms of strength, the possible failure modes for the T-stub connection are yielding of the column face, bending failure of the endplate, yielding of T-stem and bolt fracture. These are in addition to possible failure modes of the connection between the T-stub and beam (e.g. bolts in shear, ply bearing and tearing) which are well codified. A designer would also need to check for shear in the connection which is similar to that for standard T-stub connections found in most codes of practice. Detailed procedures to design the T-stub connection using Ajax ONESIDE blind bolts are outlined in Appendix A.

Design formulae for the fasteners and endplate are similar to a standard connection although consideration needs to be given to the slightly different bolt properties. Fernando (2008) suggested that the Ajax ONESIDE can be designed according to the Australian Steel Standard AS4100 (1998) as long as the appropriate bolt properties are assumed. The model to predict the yielding of the column face is discussed in detail in Section 4.6.1 as this particular failure mode is found to be the governing mode for T-stub connections (Kurobane et al., 2005) for the typical range of tube and endplate thicknesses. In terms of stiffness prediction for the T-stub connections, the component model proposed by the Eurocode 3 (European Committee for Standardisation (CEN), 2005) is utilised and is discussed in Section 4.6.2.
Blind Bolted Connections for Steel Hollow Section Columns

Figure 4-30: Summary of T-stub connection design

Step 1: Determine design actions on connection (M', T' & V') and determine design forces for each T-stem, F' (Eq. A-1)

Step 2: Check for column face yielding and pull-out action of blind bolts from tube face, revise F' from Step 1 (Eqs. A-2 & A-3)

Step 3: Check thickness of T-stem, tstem required to transmit tension force in the T-stem (Eqs. A-4 & A-5)

Step 4: Determine minimum number of blind bolts, Nbb required to carry tension force from beam flange to column face, check blind bolts for shear and combined actions (Eqs. A-6 to A-8)

Step 5: Determine minimum number of standard bolts, Nsb to transfer shear force in the T-stem (Eqs. A-9 & A-10)

Step 6: Determine T-stub endplate thickness, taking prying action into account (Eqs. A-11 & A-12)

Component method based on Eurocode3 approach

where $K_{compression} \approx 5K_{tension}$

$$K_{tension} = \frac{1}{K_{tube} + \frac{1}{K_{bolt}} + \frac{1}{K_{ep}}}$$

Refer to Eqs. 4-4 to 4-6 for $K_{bolt}$, $K_{ep}$ and $K_{tube}$ respectively

Hence, overall connection stiffness, $S_{j,ini}$:

$$S_{j,ini} = \frac{M}{\theta} = \frac{\varepsilon^2}{\frac{1}{K_{tension}} + \frac{1}{K_{comp}}} = \frac{\varepsilon^2}{2/K_{tension}}$$
4.6.1 Column face plastification – strength

The inherent flexibility of the hollow section column face is the main difference when comparing the connection to a hollow section column or an open section column (H or I section). For the connection to hollow section columns at the ultimate condition, a number of researchers have developed various versions of the column face plastification model based on yield line theory. The differences between these versions are mainly due to the different types of blind bolts being used: Yeomans (1996b; 1998) for Flowdrill and Mourad (1994) for Huck HSBB (refer Figure 4-31).

The model proposed by Yeomans (1996b; 1998) had been modified slightly and incorporated into the CIDECT Design Guide 9 (2005) for the Flowdrill blind bolting system as given in Equation 4-1:
Blind Bolted Connections for Steel Hollow Section Columns

\[
N_{pl} = \frac{f_{c,y} t_c}{2} \left[ \frac{2(h_b - d_b)}{b'} + 4 \sqrt{\left(1 - \frac{c}{b'}\right)} \right] \left( \frac{1 - \frac{c}{b'}}{1 - \frac{c}{b'}} \right) \tag{4-1}
\]

where \( N_{pl} \) = column face plastification load; \( f_{c,y} \) = column yield strength; \( t_c \) = column thickness; \( b_c \) = column width; \( d_b \) = diameter of bolt, \( b' = b_c - t_c \) and \( c = g - d_b \). Refer to Figure 4-31 for dimensions \( g \) and \( h_b \).

Another column face plastification formula has been proposed by Mourad (1994) for Huck HSBB and is given in Equation 4-2:

\[
N_{pl} = \frac{2f_{c,y} t_c^2}{1 - \beta} \left[ (\omega - \gamma) + 2\sqrt{(1 - \gamma)(1 - \beta)} \right] \tag{4-2}
\]

where \( \beta = \frac{X_b}{b} \), \( \omega = \frac{Y_b}{b} \), \( \gamma = \frac{d_b}{b'} \) and \( d_h = \) diameter of bolt hole. Refer to Figure 4-31 for dimensions \( X_b \) and \( Y_b \). Note that the diameter of the bolt hole was considered instead of the diameter of the bolt itself because the HULK HSBB has an oversized bolt hole unlike the Flowdrill which has the same bolt and bolt hole diameter.

Mago and Clifton in HERA Report R4-120 (2003) investigated the effect of column axial load on the performance of bolted moment endplate connection. It was found that once the ratio of column axial compression load \( N^* \) on column design section capacity \( \Phi N \), exceeds 0.6, the column flange resistance to tension forces transmitted through the bolts is reduced. Subsequently, Clifton et al. (2009) in HERA Report R4-142 recommended that the out of plane column flange tension capacity should be reduced by a column axial load reduction factor, \( \eta \) given in NZS 3404: Part 1 (Standards New Zealand, 1997), Equation 12.9.5.3(5), where

\[
\eta = \sqrt{1.15 - \left( \frac{N^*}{\phi N_s} \right)^2} \leq 1.0,
\]
The column axial load reduction factor $\eta$, should be applied into Equations 4-1 and 4-2.

From the FE analyses of specimens S1 and S2 reported in Section 4.5, extensive panel yielding was observed instead of the formation of distinct yield lines. The panel yielding from the FE model can be simplified to represent yield lines as shown in Figure 4-32. The column face plastification mechanism shown in Figure 4-32 is similar to that given by Mourad (1994). The mechanism was formed at an applied load of 121kN for specimen S1 and 135kN for specimen S2 in the FE model; the higher value for S2 being achieved because of the addition of bolt sleeves. The plastification load from FE model was noted when the entire panel along the expected yield lines on the tube face has yielded.

From Equation 4-1, the column face plastification load ($N_{pl}$) based on the CIDECT Design Guide 9 is 101kN (using bolt diameter) and 93kN (using oversized bolt hole diameter) whereas Equation 4-2, based on the work by Mourad (1994), estimates a value of 107kN. Hence the latter gives a closer approximation to the FE prediction for specimen S1. One reason that the prediction by Mourad (1994) is closer to the FE results than the CIDECT formula is that the HUCK HSBB system is closer to the ONESIDE blind bolts than the Flowdrill blind bolting system.

The checks for other possible failure modes such as punching shear of bolts through the column face is provided in CIDECT Design Guide 9 (2005), and bolt fracture can be designed in accordance with AS 4100 (1998).
4.6.2 Component model – initial stiffness

The initial stiffness of the blind bolted T-stub connection can be predicted based on the classical component method approach proposed by Eurocode 3 (2005). Based on the FE models in Section 4.5.4, the stiffness of the tension and compression regions can be assumed to be independent of each other. The tension region of the connection is comprised of three springs in series representing the following stiffnesses: the blind bolt stiffness in tension ($K_{bolt}$), the endplate bending stiffness ($K_{ep}$) and the column flange bending stiffness ($K_{tube}$) as illustrated in Figure 4-33. The overall stiffness of the joint in tension, $K_{tension}$ can be expressed as:

$$K_{tension} = \frac{1}{\frac{1}{K_{tube}} + \frac{1}{K_{bolt}} + \frac{1}{K_{ep}}}$$

Each of these component stiffnesses is discussed below.
4.6.2.1 Blind bolt stiffness in tension, $K_{bolt}$

The stiffness of the blind bolt in tension can be predicted using the design model given by Eurocode 3 (2005) where

$$K_{bolt} = 1.6 \frac{EA_s}{L_{bolt}}$$  \hspace{1cm} (4-4)

where

- $E = \text{Young's modulus for steel, 200 000MPa,}$
- $A_s = \text{tensile area of bolt,}$
- $L_{bolt} = \text{bolt elongation length, given by}$

$$L_{bolt} = t_c + t_{ep} + 2t_w + \frac{t_{bh} + t_{nut}}{2}$$  \hspace{1cm} (4-4a)

where

- $t_c$ and $t_{ep}$ are the thicknesses of tube and endplate respectively, and $t_w$, $t_{bh}$ and $t_{nut}$ are the thicknesses of the blind bolt washers, bolt head and nut respectively.

4.6.2.2 Endplate bending stiffness, $K_{ep}$

The stiffness of the endplate can be predicted using a cantilever analogy. The endplate is assumed to be a cantilever beam with a fixed end at the intersection of endplate-stem. A point load is assumed to act on the cantilever at the bolt...
Blind Bolted Connections for Steel Hollow Section Columns

location (refer Figure 4-34a). Hence, the stiffness of the endplate can be calculated using Equation 4-5.

\[ K_{ep} = 0.5 E \frac{b_{eff} t_{ep}^3}{m^3} \]  \hspace{1cm} (4-5)

where

- \( b_{eff} \) = effective width of endplate (refer (Faella, Piluso, & Rizzano, 2000)), is taken as the actual width of the endplate for the configuration used in the experiments
- \( m \) = distance from intersection of stem and endplate to bolt location

![Figure 4-34: Component model for T-stub connection](image)

\[ S_{j,ini} = \frac{M}{\theta} = \frac{z^2}{\sqrt{1/K_{tension}}} + \frac{1}{\sqrt{K_{compression}}} \approx \frac{z^2}{1.2/K_{tension}} \]

where \( K_{compression} \approx 5 K_{tension} \) (refer Table 4-5)

4.6.2.3 Column face bending stiffness, \( K_{tube} \)

The flexible column face bending stiffness is predicted on the basis of simplified finite element modelling performed by Mourad (1994). The stiffness of the column face in bending, \( K_{tube} \) is given by Equation 4-6.

\[ K_{tube} = \frac{E t_e^3}{12(1-v^2)} \frac{1}{R \cdot \gamma (b_c - 2t_c)^2} \]  \hspace{1cm} (4-6)
where

\[ b_c = \text{width of column}, \ \nu = \text{Poisson’s ratio for steel}, 0.3 \]
\[ \gamma_s = \text{deflection coefficient (obtained from design charts)} \]
\[ R^* = \text{reduction factor due to corner restraints (obtained from design charts)} \]

4.6.2.4 Validation of component model and sensitivity analysis

To validate the component model for predicting the initial stiffness, Equations 4-3 to 4-6 were applied to configurations replicating those of test specimens S1 and S2. The results from the component model and corresponding FE results are summarised in Table 4-3. The theoretical predictions give a lower and hence conservative estimate compared with the FE models, with an error margin of 15%. For Specimen S1 (without sleeves), the component method underpredicts the FE model results by 9% and experimental results by 11%. For Specimen S2, where sleeves with very tight tolerance were present, the component model underpredicts the FE model by 14% and experimental results by 20%. The main reason for the difference between the component model and FE or experimental results is the significant simplification of the behaviour of the various components; in particular the endplate and tube face yield models.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Theoretical calculations</th>
<th>FE model</th>
<th>Ratio</th>
<th>Experimental</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number</td>
<td>(K_{ep})</td>
<td>(K_{tube})</td>
<td>(K_{bolt})</td>
<td>(K_T)</td>
<td>(K_{FE})</td>
</tr>
<tr>
<td>S1</td>
<td>55</td>
<td>81</td>
<td>1322</td>
<td>32</td>
<td>35</td>
</tr>
<tr>
<td>S2</td>
<td>55</td>
<td>81</td>
<td>1322</td>
<td>32</td>
<td>37</td>
</tr>
</tbody>
</table>

Note: Units for stiffness, \(K\): kN/mm
\(K_T\) is the component model assembly for tension region

To validate the component model further, a large sensitivity analysis was performed on typical connection configurations. A total of 11 FE models for different column sizes, endplate thickness and geometric configurations were modelled to validate the component method in the tension region. Table 4-4 compares the stiffnesses from the FE predictions to those from the component model.
model (also referred to as theoretical stiffness) for the various case studies. The component method predictions are in the range of ± 20% of the FE model. This gives confidence that the component model, despite its simplicity, is able to predict the initial stiffness of the T-stub connection in the tension region reasonably well for the typical cases covered by the sensitivity analysis.

### Table 4-4: Comparison of component model to FE prediction of tension stiffness

<table>
<thead>
<tr>
<th>Model</th>
<th>Tube size</th>
<th>Tube thk (mm)</th>
<th>n (mm)</th>
<th>m (mm)</th>
<th>Endplate thk (mm)</th>
<th>$K_t$ (kN/mm)</th>
<th>$K_{FE}$</th>
<th>$K_{component}$</th>
<th>$K_{component}/K_{FE}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>150×150×6 SHS</td>
<td>150</td>
<td>6</td>
<td>60</td>
<td>65</td>
<td>10</td>
<td>35</td>
<td>32</td>
<td>0.91</td>
</tr>
<tr>
<td>2</td>
<td>150×150×9 SHS</td>
<td>150</td>
<td>9</td>
<td>60</td>
<td>65</td>
<td>20</td>
<td>62</td>
<td>64</td>
<td>1.04</td>
</tr>
<tr>
<td>3</td>
<td>150×150×9 SHS</td>
<td>150</td>
<td>9</td>
<td>60</td>
<td>65</td>
<td>10</td>
<td>49</td>
<td>48</td>
<td>0.98</td>
</tr>
<tr>
<td>4</td>
<td>125×125×5 SHS</td>
<td>125</td>
<td>5</td>
<td>60</td>
<td>65</td>
<td>10</td>
<td>32</td>
<td>28</td>
<td>0.88</td>
</tr>
<tr>
<td>5</td>
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<td>60</td>
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<td>10</td>
<td>48</td>
<td>44</td>
<td>0.92</td>
</tr>
<tr>
<td>6</td>
<td>125×125×5 SHS</td>
<td>125</td>
<td>5</td>
<td>60</td>
<td>65</td>
<td>20</td>
<td>61</td>
<td>60</td>
<td>0.99</td>
</tr>
<tr>
<td>7</td>
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<td>60</td>
<td>65</td>
<td>10</td>
<td>38</td>
<td>34</td>
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<tr>
<td>8</td>
<td>89×89×5 SHS</td>
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<td>5</td>
<td>40</td>
<td>40</td>
<td>10</td>
<td>51</td>
<td>60</td>
<td>1.17</td>
</tr>
<tr>
<td>9</td>
<td>89×89×5 SHS</td>
<td>89</td>
<td>5</td>
<td>40</td>
<td>60</td>
<td>10</td>
<td>31</td>
<td>31</td>
<td>1.01</td>
</tr>
</tbody>
</table>

Note: Refer to Figure 4-34a) for dimensions m and n

The same 11 FE models were then rerun with their stems loaded in compression. The stiffness of each connection under compression is compared with that under tension and the results are summarised in Table 4-5. The compression region of the T-stub connection was found to be greater than five times the stiffness of tension region for the cases investigated in Table 4-5 except for Model 5 in which a thick tube and thick endplate are both present. Therefore, for simplicity it can be assumed that the overall connection stiffness (combining tension and compression regions) is $0.8K_t$, i.e. 80% of the tension stiffness. This simplification would mean that a designer would only need to calculate the tension stiffness of
the T-stub connection to estimate the stiffness of the whole joint. Hence, the component model to predict the stiffness of the entire beam-to-column T-stub connection is illustrated in Figure 4-34b, assembling the contribution from the tension and compression regions of the connection. The component model developed in this section forms the basis for other more advanced component models in subsequent chapters to determine the stiffness of the whole connection.

![Table 4-5: Comparison of T-stub connection stiffness in compression region to tension region](image)

<table>
<thead>
<tr>
<th>Model</th>
<th>Tube size</th>
<th>Tube thk (mm)</th>
<th>n (mm)</th>
<th>m (mm)</th>
<th>Endplate thk (mm)</th>
<th>$K_{tension}$ (kN/mm)</th>
<th>$K_{compression}$ (kN/mm)</th>
<th>$K_{compression}/K_{tension}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>150×150×6 SHS</td>
<td>60</td>
<td>65</td>
<td>10</td>
<td>35</td>
<td>247</td>
<td>7.1</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>150×150×6 SHS</td>
<td>60</td>
<td>65</td>
<td>20</td>
<td>62</td>
<td>359</td>
<td>5.8</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>150×150×6 SHS</td>
<td>60</td>
<td>65</td>
<td>10</td>
<td>49</td>
<td>266</td>
<td>5.4</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>150×150×9 SHS</td>
<td>60</td>
<td>65</td>
<td>10</td>
<td>55</td>
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<td>60</td>
<td>65</td>
<td>20</td>
<td>124</td>
<td>511</td>
<td>4.1</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>125×125×5 SHS</td>
<td>60</td>
<td>65</td>
<td>10</td>
<td>32</td>
<td>221</td>
<td>6.9</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>125×125×5 SHS</td>
<td>60</td>
<td>65</td>
<td>10</td>
<td>48</td>
<td>243</td>
<td>5.1</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>125×125×5 SHS</td>
<td>60</td>
<td>65</td>
<td>20</td>
<td>61</td>
<td>333</td>
<td>5.5</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>125×125×5 SHS</td>
<td>60</td>
<td>45</td>
<td>20</td>
<td>68</td>
<td>355</td>
<td>5.2</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>89×89×5 SHS</td>
<td>40</td>
<td>40</td>
<td>10</td>
<td>51</td>
<td>276</td>
<td>5.4</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>89×89×5 SHS</td>
<td>40</td>
<td>60</td>
<td>10</td>
<td>31</td>
<td>272</td>
<td>8.8</td>
<td></td>
</tr>
</tbody>
</table>

Note: Refer to Figure 4-34a) for dimensions m and n

### 4.7 Summary and conclusions

T-stub blind bolted connections to unfilled SHS columns were tested separately in both tension and compression. The experimental results were then used to validate detailed FE models. In addition approximate design models were suggested for application by designers. The findings from this work can be summarised as below:
1) T-stub connection using Ajax ONESIDE blind bolts was developed and tested. The experimental and analytical results presented in this chapter provide a basis for further study on developing a range of blind bolted moment connections for hollow section columns, which exhibit higher levels of initial stiffness to achieve rigid status.

2) Bolt sleeves are effective in reducing bending of the blind bolts, thereby improving the strength and stiffness of the connection. It is recommended that the sleeves should always be used as part of the ONESIDE assembly. Their efficiency is dependent on the tolerance between the outer diameter of the sleeves and bolt hole. The tighter the tolerance, the more efficient the sleeves are in reducing bending of the bolts and in reducing slip between the endplate and the tube face. The sleeve tolerance however does not have a significant influence on the initial stiffness of the connection.

3) The results show that the T-stub blind bolted connection behaves in a semi-rigid manner according to the Eurocode 3 specification. Based on finite element modelling, increasing endplate thickness can enhance the stiffness of the connection. However this is only true for an endplate thickness up to approximately 20mm, beyond which the stiffness of the connection is dominated by the flexibility of the column face.

4) The load predicted by the column face plastification formula that has been derived from yield line theory shows good agreement with the results from the non-linear finite element model.
5) The 3-D non-linear finite element modelling developed for the T-stub tests can replicate the behaviour of the test specimens with very good accuracy.

6) The component model developed here is able to predict the initial stiffness of the connections with reasonable accuracy.

7) The initial stiffness of the T-stub connection loaded in compression can be conservatively taken as five times the initial stiffness of the tension region for the cases investigated in this chapter. Thereby for simplicity, the initial stiffness of the overall beam-to-column T-stub connection can be taken as 80% of the initial stiffness in the tension model.
Chapter 5

Blind Bolted Collar Plate Connection

5.1 Introduction

In Chapter 4 a typical face connection, the T-stub connection, was discussed. The T-stub connection has a stiffness in the semi-rigid range. Increasing endplate thickness can evidently enhance the stiffness of the connection. However this is only true up to a certain endplate thickness, beyond which the stiffness of the connection is dominated by the flexibility of the column face. The inherent flexibility of the hollow section column face compromises the face connection stiffness. The literature review in Chapter 2 illustrated connections with welded components near corners of the tube to transfer load more directly to the side walls, away from the flexible front face of the tube. A similar idea is explored in this chapter whereby a collar plate is blind bolted to the side walls of the tube to transfer loads directly from the beam to the side walls, thus avoiding loading on the flexible column face. This connection is referred to herein as the collar plate connection and is proposed as an alternative to the T-stub connection. This
chapter aims to investigate the behaviour of the blind bolted collar plate connection and explore parameters that affect its stiffness.

5.2 Experimental program

5.2.1 Test specimen

A blind bolted collar plate specimen was constructed and tested. The test specimen was constructed to simulate the tension region of a beam to column connection. Details of the specimen are shown in Figure 5-1. The specimen is comprised of a 150×150×6mm Grade 350 SHS column and a collar plate of 20mm thickness Grade 300. A thick collar plate was chosen to eliminate flexibility of the collar plate. Four Grade 8.8 M16 ONESIDE bolts with sleeves were used to connect the collar plate to the steel tube. The sleeves used in this experiment had tight tolerances to the bolt holes, and the effect of varying this tolerance was investigated in the FE sensitivity studies discussed in Section 5.4. The bolts in the specimen were tensioned to a snug tight condition.

Figure 5-1: Details of the test specimen and instrumentation
5.2.2 Experimental setup and instrumentation

An illustration of the instrumentation for the test specimen is shown in Figure 5-1. Linear Variable Displacement Transducers (LVDTs) were mounted on the specimen relative to the ground to record the deformation of the collar plate, tube face and relative movement between the collar plate and tube side wall. Strain gauges were placed at the bottom face of the SHS tube to measure the strains of the bottom tube face. Loading was applied to the test specimen via the stem of the collar plate by gently increasing the displacement up to failure. The specimen setup is shown in Figure 5-2.

![Figure 5-2: Collar plate connection specimen setup](image)

A digital photogrammetry survey technique with accuracy of 0.03mm was also employed in this test. Photogrammetry targets were placed on the collar plate and around the hollow section tube. An initial survey was taken prior to load being applied and subsequent surveys were taken at load intervals of 50kN up to 350kN. Thereafter the specimen was loaded to failure.
5.3 Test results

5.3.1 Collar plate connection strength and failure mode

A 20mm thick collar plate was employed in the test to reduce the flexibility and potential for failure of the collar such that the focus of the investigation was on the SHS tube. In typical low-rise construction in Australia the columns tend to be smaller than the beams. Hence, the strength hierarchy at the joint is such that the column is expected to reach its capacity before the connection or the beam. As discussed in Chapter 4 earlier, an initial upper bound estimate of the tension force transferred by the collar plate can be made by dividing the column section bending capacity by the beam depth. For a 150x6mm Grade350 SHS column and a 300mm deep beam, this would correspond to a tension force of 170kN. The connection would also have to comply with deformation limits at serviceability and ultimate limit states.

At low levels of applied load, there was not much observable deformation in the specimen. As the load was gradually increased to the collar stem, the collar plate slowly separated from the tube face. While this occurred, the face of the collar plate remained flat; no bending of the collar plate was observed. At a load of 400kN, the LVDT on the collar plate recorded a rapid increase of displacement while the load remained constant. Upon inspection, it was observed that the bottom face of the tube had buckled in compression while the connection remained intact. Figure 5-3 shows the tube after failure has occurred with buckling of the tube bottom face and elongation of the bolt holes due to localised bearing of the bolts on the tube side wall.
5.3.2 Deformation of collar plate

Figure 5-4 shows the displacements recorded by LVDTs on the collar plate. At a load of 400kN, when the bottom face of the tube buckled, considerable deformation was recorded by the LVDTs. The deformation limits recommended by the International Institute of Welding (IIW) (1989) are also included in Figure 5-4. The serviceability limit state deformation of 1% column width corresponded to an estimated load of 140kN in the specimen, while a load of 260kN was estimated at the ultimate limit state deformation of 3% column width. The maximum load on the connection will not exceed 170kN as discussed previously in Section 5.3.1 (i.e. governed by the column capacity). Hence the connection will remain stiff throughout the expected full loading range.
5.3.3 Relative movement between tube side wall and collar side plate

The relative movements between centerline of tube side wall and top of collar plate are measured by LVDTs shown in Figure 5-2. Although there was a slight difference in the initial slip load (NW and SW bolts started slipping after 50kN while NE and SW bolts started slipping at 20kN), the bolts displayed a similar trend in the load versus relative movement behaviour as shown in Figure 5-5. At the estimated ultimate load of 170kN (refer to earlier prediction from Section 4.2), the average relative movement between the tube and the collar side plate was approximately 0.5 mm. This is remarkably stiff behaviour but it can be explained by the fact that there was very little difference in the slip of the bolts due to the tight tolerances for the holes, sleeves and bolts.
5.3.4 Strains

Three strain gauges were attached to the bottom face of the tube. Similar levels of strain were recorded at all three strain gauges (shown in Figure 5-6). Yielding in the bottom tube face commenced at a load of about 300kN. Beyond an applied load of 350kN strains in the bottom face increased rapidly, indicating that the bottom face of the tube was highly stressed.

5.3.5 Comparison of collar plate connection to T-stub connection

A simple comparison is made in Figure 5-7 between the load displacement behaviour for the collar plate connection and T-stub connection. It can be seen that the collar plate connection is far stronger and stiffer than the T-stub connection which shows that connecting to the sides of the hollow section columns has good potential.
Figure 5-6: Strain at underside of tube

Figure 5-7: Comparison of load displacement curve for collar plate connection and T-stub connection
5.4 Finite element analysis

A three-dimensional finite element model was created in ANSYS 11.0 to represent the blind bolted collar plate connection to the unfilled SHS column. The FE model takes into account material and geometric non-linearities and complex contact interactions between various surfaces. Surface to surface contact elements with friction coefficient of 0.15 were employed in the FE model. Taking advantage of symmetric conditions along the longitudinal and transverse planes, only a quarter of the test specimen was modelled. A half-symmetry finite element model of the collar plate connection is shown in Figure 5-8 (for ease of visualisation). Load was applied to the stem of the collar plate and a line of support was provided at the end of the tube simulating support conditions during the experiment.

![Figure 5-8: 3-dimensional FE model (half symmetry)](image)

5.4.1 Comparison between the FE analysis and test results

Comparisons between the deformations recorded using the photogrammetry surveys of the collar plate during the test and the prediction from the FE model are made in Figure 5-9. In this figure, Experiment-S and N denote a
Blind Bolted Connections for Steel Hollow Section Columns

photogrammetry target on the southern and northern sides of the specimen respectively. The prediction from the FE model of the collar plate deformation agrees well with the experimental results. There is a slight discrepancy at loads of 50kN, where the experimental results appear to be stiffer than the FE prediction. This is likely due to slight pretension in the bolts from tightening during set up of specimen. The initial slight pretension has not been considered in the FE model.

![Figure 5-9: Comparison of FE and photogrammetry results for collar plate displacement](image)

The results from FE model were further validated with photogrammetry targets along the longitudinal centerline of the collar plate for different stages of applied load. Comparisons of results are shown in Figure 5-10; the FE model replicates the collar plate deformed profile very well.
5.4.2 Sensitivity analysis

The FE model has shown an excellent correlation with experimental results and is now used to perform sensitivity analysis. Two parameters are considered in the
sensitivity analyses presented in this chapter. These are collar plate thickness and size of bolt sleeves as discussed below.

5.4.2.1 Collar plate thickness

A thick collar plate of 20mm was adopted in the experiment. A sensitivity analysis was carried out to investigate the effect of having a thinner collar plate. The collar plate thickness was reduced from 20mm to 10mm in the FE model. All other aspects of the model remained the same. A comparison of collar plate displacements for both models is shown in Figure 5-11. Both models are shown to have a very similar initial stiffness and ultimate capacity (this was governed by buckling of column face). This indicated that transferring loads to the sides of the column relied mostly on shear and bearing actions of the bolts. A collar plate thickness of 10mm is a better match for a 6mm thick column in practice and gives a comparable performance to the thick 20mm collar plate which was used in the test.

![Figure 5-11: Sensitivity analysis between 10mm and 20mm collar plate](image)
5.4.2.2 Tolerance between bolt hole and outer diameter of sleeve

The tolerance between the outer diameter of the sleeves and the bolt hole was also investigated. The tight tolerance between the outer diameter of sleeves and bolt holes provided a very tight fit for the connection and this is not practical for most cases. A 2mm standard bolt hole tolerance, which is typical for bolted connections with bolt size smaller than M24, was employed in the sensitivity analysis. With this higher level of bolt hole tolerance, tightening the bolts to the snug tight condition would not be adequate since early slip would result in a large reduction in the initial stiffness. The bolts were pretensioned up to 70% of yield load (70kN) in the model. The model with 2mm tolerance appeared to have a similar initial stiffness and ultimate strength to the model with tight tolerance with snug tight bolts as shown in Figure 5-12. The slip load depends on the level of bolt pretension and the coefficient of friction between the slip surfaces. The results of the modelling indicate that the sleeves adopted in the experiment with tight tolerance provided an upper bound to the stiffness of the collar plate connection.

![Figure 5-12: Comparison of load-displacement curves for different sleeve tolerance](image)

Figure 5-12: Comparison of load-displacement curves for different sleeve tolerance
5.4.3  Moment rotation curves – comparison to Eurocode 3 specifications

In order to assess the behaviour of the full connection, a FE model of the entire connection (including tension and compression zones) was developed as shown in Figure 5-13. Parameters used in the full FE model are the same as the specimen tested in the experiment, with a collar plate thickness of 20mm and a tight tolerance between the bolt sleeves and the bolt holes. The initial rotational stiffness, $S_{\text{ini}}$, of the connection is classified in accordance with Eurocode 3 Part 1-8 (European Committee for Standardisation (CEN), 2005).

Assuming a 6m beam length which is common for low rise residential structures, the classification limit based on the properties of a 310UB32 beam is plotted on Figure 5-14. It can be seen that the stiffness of the collar plate connection is very close to the rigid classification for braced frames. This gives an indication that connecting to the sides of hollow section columns has the capacity to achieve a rigid connection for braced frames. The collar plate connection is approximately five times stiffer than the T-stub connection. It should be noted that the slip between the beam flange and the collar plate stem has not been considered here. Also, the beam flange and collar tension/bearing flexibility has been disregarded.
Rotation, $\theta = (\Delta_t + \Delta_c) / h$

- $\Delta_t$ – displacement in tension region
- $\Delta_c$ – displacement in compression region

- $\Delta_t$ measured at this point relative to tube centerline
- $\Delta_c$ measured at this point relative to tube centerline

Rotation, $\theta = (\Delta_t + \Delta_c) / h$

$h$ = lever arm

Figure 5-13: FE model of full beam-to-column collar plate connection

- $S_{j, \text{rigid (braced)}} = 16850 \text{kNm/rad}$
- $S_{j, \text{overall}} = 15000 \text{kNm/rad}$

Figure 5-14: Moment rotation curve for collar plate connection from FE model
5.5 **Summary and conclusions**

A collar plate blind bolted connection to an unfilled SHS column in the tension region has been tested in tension. The findings from the test and FE modelling are summarised below:

1) The results show that the collar plate blind bolted connection has the potential to be classified as a rigid connection for a braced frame system according to the Eurocode 3 specifications. The stiffness of the connection is not compromised by the flexible hollow section column face deformation normally observed in a typical face connection.

2) The tight tolerance provided by the bolt sleeves contributes to the connection stiffness. For industry practice it is recommended that standard bolt hole tolerance of 2mm is provided between the outer diameter of the sleeves and bolt hole. The bolt should be pretensioned to prevent slip under serviceability loading.

3) 3-D non linear finite element modelling developed for the collar plate connection can replicate the behaviour of the test specimen successfully.

4) Based on a sensitivity analysis with finite element modelling, a connection with a collar plate thickness of 10mm has a similar initial stiffness and ultimate capacity to the connection with a 20mm thick collar plate.

5) Similarly a FE model with a 2mm standard bolt hole tolerance for bolt sleeves with bolts pretensioned to 70% yield load, displayed a similar initial stiffness to the model with a tight tolerance for the sleeves and with snug tight bolts.
6) The collar plate connection although found to be much stiffer than the T-stub connection requires extensive fabrication work to weld pieces of plates together to form the collar and the need to fabricate different collars to suit different column sizes. A simpler alternative to the collar plate connection and a more versatile arrangement is required for a more feasible side connection which will be investigated in the next chapter.
Chapter 6

Connection to sides of hollow section columns

6.1 Introduction

In Chapter 5 a collar plate connection was tested as an alternative to the T-stub connection which is a typical face connection as discussed in Chapter 4. The collar plate connection which connects beam flanges to the sides of the hollow section column has shown promising improvements in terms of stiffness and strength of the connection. One shortcoming of the collar plate connection would be the extensive fabrication work that is needed to weld pieces of plates together to form the collar and the need to fabricate different collars for different column sizes. In this chapter another alternative to the collar plate connection is developed whereby minimal fabrication work is needed for this new connection configuration, referred to herein as the blind bolted channel side plate connection. This connection as for the collar plate, connects the beam flanges to the sides of hollow section columns. This is a simple alternative connection to the collar plate connection.
The channel side plate connection proposed in this chapter incorporates a SHS column, open section beam, side plates which are cut from standard flats, and channels which are cut from the same type of SHS column. Channels are first connected to the top and bottom flanges of the open section beam with standard structural bolts. Straps or side plates are then bolted to the legs of the channel sections with standard structural bolts on one side. On the opposite side, the straps or side plates are first blind bolted to the SHS column. Finally the various components are brought together and bolted in place (refer Figure 6-1). This configuration uses only bolts on site with no need for welding at all. All bolts must be fully pretensioned to achieve good initial stiffness for the connection.

Figure 6-1: Procedure to assemble connection

In order to assess the feasibility and performance of such a connection, a full scale connection test was carried out as outlined in Section 6.2, with the results described in Section 6.3. An FE model of the connection was developed to predict the behaviour and also to undertake a sensitivity analysis as detailed in Section 6.4. Finally a design model was developed for this connection to estimate its stiffness and strength as outlined in Section 6.5.
6.2 Experimental program

6.2.1 Test specimen

A full scale beam-to-column channel side plate connection was developed and tested as shown in Figure 6-2. Details of the specimen are shown in Figure 6-3. The specimen comprised a Grade 300, 310UB32 universal beam (approximately 310mm deep, 32kg/m section) connected to a 150×150×6mm Grade 350 SHS column. The 6mm channels which were cut from the SHS column section were bolted with standard M16 structural grade 8.8 bolts to the beam flanges. The legs of the channels were then bolted to 12mm thick side plates and the side plates were connected to the side faces of the SHS column with structural grade 8.8 M16 ONESIDE blind bolts. All of the bolts in the specimen were fully pretensioned. In order to determine the exact pretension load in the blind bolts for the purpose of the test, a special type of strain gauge (TML strain gauge BTM-6C) was embedded inside the shanks of these bolts. The blind bolts were pretensioned according to the readings from these strain gauges while a torque wrench was used to apply a torque of 30kg.m for the remaining standard M16 structural bolts as recommended by the manufacturer.

6.2.2 Experimental setup and instrumentation

A hydraulic actuator was used to apply horizontal load at the cantilever end of the beam as shown in Figure 6-3. The SHS column was restrained at both ends and tied down to the strong floor. Displacement transducers (LVDTs) were used to measure relative movements between the channels, side plates, beam and the tube. An illustration of the instrumentation used to monitor the behaviour of specimen is shown in Figure 6-4.
Figure 6-2: Photo of specimen and zoomed in view of connection

Figure 6-3: Test specimen setup (dimensions are in mm)
Digital photogrammetry was also employed in this test. Photogrammetry targets were placed on the channels, side plates, beam and SHS tube to obtain the overall deformation profile.

### 6.3 Test results

Loading, under displacement control, at the end of the cantilever beam was gradually increased until the test specimen failed. As the load was increased, the beam started to tilt in the direction of the applied load. Slip occurred at an applied load of approximately 35kN as can be observed from the graphs in Figure 6-7 to Figure 6-12. However, the load versus displacement behaviour of the connection started to deviate from linearity at a load of approximately 20kN. At higher loads, the column started to deform in bending; bearing of the beam flange against the face of the channel was also observed. A large overall deflection of the beam occurred after the channels yielded and then deformed plastically, resulting...
in large rotations in the connection region. An actuator applied load of 140kN corresponds to a shear force of 100kN in each bolt on the beam flange which is the code-estimated shear capacity of these M16 structural bolts. At this load level failure of the bolts was imminent, and the test was stopped at this point. This is well beyond the expected working range if the connection in practice.

The deformed shape of the specimen at an applied load of 140kN is shown in Figure 6-5. The beam flanges buckled in the compression zone of the connection and both channels in the tension and compression regions were highly distorted. Significant distortions were observed due to bolt shanks bearing against the walls of the tube and channels at the bolt holes (refer Figure 6-6). The column itself did not undergo much deformation except for localised bearing around the bolt holes. The face of the column remained perfectly flat unlike the flexible column face deformation observed in a typical front face connection. By connecting to the sides of the SHS column, the flexible column face deformation which is unfavourable to connection stiffness could be avoided.

![Overall deformed shape at failure](image-url)
6.3.1 Overall beam deformation – LVDT 1

Figure 6-7 shows the overall beam deflection; the connection had high initial stiffness up to a load of 20kN followed by a well-defined slip at 35kN which corresponded to a force of 15kN in each bolt at the start of the slip to 25kN in each bolt when the well defined slip occurred, with a further reduction in stiffness at higher loads due to yielding of the channel sections. The additional flexibility in post-slip stiffness can be attributed to the continued slippage of the bolts before bearing occurs while the connection was loaded. The unloading stiffness and reloading stiffness of the connection at a load of 140kN were similar and were approximately 70% of the initial stiffness.
6.3.2 Relative moment between beam and channel – LVDT 2

A similar trend was observed for relative movement between the beam and channel in Figure 6-8. Bolts connecting the beam and channel slipped at an applied load of 35kN. At approximately 105kN, deformation increased at a faster rate indicating that plastic deformation was occurring in the channel.
Relative movement between tube and side plates in the tension and compression regions appeared to be quite symmetrical from Figure 6-9. Slip occurred at the same time for the blind bolts and the standard bolts as shown in Figure 6-8 Figure 6-9 (same slip load in both graphs).
6.3.4 Relative moment between tube and side plate – LVDTs 5 – 9

Figure 6-10 and Figure 6-11 show the relative movement between the beam flange and the tube in both the compression and tension regions. The compression region experienced a lesser amount of overall slip compared to the tension region. This is attributed to the location of bolts within the holes in the initial setup position. Similar trends were observed in both regions. Figure 6-12 indicates that there was virtually no relative movement in the horizontal direction between the beam and the tube until slip occurred.
Figure 6-10: Load vs. displacement for LVDTs 5 and 6 – beam flange to tube top face, $\Delta y$

Figure 6-11: Load vs. displacement for LVDTs 7 and 8 – beam web to tube side face, $\Delta y$
6.3.5  Moment rotation response

The load-displacement curves from the experimental results (LVDTs 7 and 8) were converted to a moment-rotation curve for a typical 310UB32 beam as shown in Figure 6-13. The initial rotational stiffness of the connection $S_{j,ini}$ is classified in accordance with Eurocode 3 Part 1-8 (European Committee for Standardisation (CEN), 2005).

Assuming a 6m beam length which is common for low rise residential structures and consistent with the beam size that has been used here, the classification limits based on the properties of a 310UB32 beam are plotted on Figure 6-13. It can be seen that the initial stiffness of the channel side plate connection can be classified as rigid for braced frames based on the Eurocode 3 classifications. This level of stiffness was achieved up to a moment of 20kNm (which is approximately 36% of the SHS column moment capacity) beyond which slip commenced and the connection became more flexible.
A brief comparison was made to the stiffness of the T-stub connection tested in Chapter 4. For the same size SHS column and beam, the stiffness of the T-stub connection was estimated to be 3500kNm/rad. The channel side plate connection is approximately five times stiffer than the T-stub connection. Finite element sensitivity studies have been carried out in Section 6.4.2 to investigate parameters that will affect the initial stiffness and slip load of the channel side plate connection.

![Graph showing moment rotation response from LVDTs 7 and 8](image)

**Figure 6-13: Moment rotation response from LVDTs 7 and 8**

Figure 6-14 shows the comparison between the deformation of collar plate connection (from Figure 5-4) and channel side plate connection. The deformation is plotted against the tension load in beam flange for a direct comparison between the two connections. It can be seen from Figure 6-14 that both the connections reached the serviceability limit state deformation of 1% column width at approximately the same load (130kN), although the channel side plate connection has a higher initial stiffness due to the pretensioning of all bolts in the connection. Beyond the 130kN load in the beam flange, the channel side plate connection...
starts to slip, and hence the ultimate limit state deformation of 3% column width is reached at a lower load of 140kN compared to 260kN in the collar plate connection. The collar plate connection has does not exhibit any slip due to the very tight tolerances between the blind bolt sleeves and bolt hole.

![Comparison of deformation limits between collar plate and channel side plate deformation (LVDT 5)](image)

6.4 Finite element analysis

A three-dimensional finite element (FE) model was created using the general purpose software ANSYS to represent the tested connection. The FE model takes into account material and geometric non-linearities and complex contact interactions between the various elements. Contact elements are adopted between the underside of the channel and the beam flange, the side plate and channel leg, and between various surfaces of the bolt components (this was discussed earlier in Chapter 4). Surface to surface contact elements with friction coefficient, $\mu$, of 0.15 (which is the commonly adopted $\mu$ for painted surfaces) were employed in the FE
Taking advantage of symmetric conditions along the longitudinal plane, one half of the test specimen was modelled. A full FE model of the channel side plate connection is shown in Figure 6-15 for ease of visualisation. Load was applied at the shear centre of the beam web and a line of supports was provided at the top and bottom faces of the tube in both the tension and compression regions to simulate support conditions during the experiment.

The PRETS179 element was used to create a pretension section in each bolt. A pretension load of 95kN (full pretension load for M16 structural bolts) was applied to each of the pretension nodes in the same orientation as that of the bolts. The preloading on the bolts was applied in the first load step; it produced an initial clamped displacement which was locked by the FE program, and subsequent load on the beam was applied in the second load step.

The FE model was used to predict initial stiffness of the connection, slip between various elements and deformation of all elements including stretching of side plates and distortion of channels. The material properties of the SHS column, beam, channels, side plates and bolts were described by bilinear stress-strain curves based on the von Mises yield criteria with rate independent isotropic work hardening. The yield stress and tangent modulus of the various elements are summarised in Table 6-1. The various components of the blind bolts were modelled. The inner diameter of the sleeve was 16.5mm and its outer diameter was 22.5mm leaving a gap of 0.5mm to the 16mm diameter bolt shank and 1.5mm to the 24mm diameter bolt holes.

<table>
<thead>
<tr>
<th>Item</th>
<th>Yield Stress</th>
<th>Elastic modulus, $E_o$</th>
<th>Tangent modulus, $E_{tg}$</th>
</tr>
</thead>
</table>

Table 6-1: Material properties for bilinear model (characteristic values)
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<thead>
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<th></th>
<th>(MPa)</th>
<th>(MPa)</th>
<th>(MPa)</th>
</tr>
</thead>
<tbody>
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<td>200000</td>
<td>0.01E₀ = 2000</td>
</tr>
<tr>
<td>Channels</td>
<td>350</td>
<td>200000</td>
<td>0.01E₀ = 2000</td>
</tr>
<tr>
<td>Beam</td>
<td>300</td>
<td>200000</td>
<td>0.01E₀ = 2000</td>
</tr>
<tr>
<td>Side plates</td>
<td>300</td>
<td>200000</td>
<td>0.01E₀ = 2000</td>
</tr>
<tr>
<td>Bolts</td>
<td>640</td>
<td>200000</td>
<td>0.01E₀ = 2000</td>
</tr>
</tbody>
</table>

Figure 6-15: FE model of side plate connection which simulates the experimental setup

6.4.1 Comparison to test results

The FE model was validated with experimental results from LVDT records and photogrammetry surveys.

Figure 6-16 to Figure 6-20 show the comparison between experimental results and FE predictions for various LVDTs. It can be seen from the graphs that the FE model gives a very good prediction of the initial stiffness and overall deformation profile. The FE model appears to over-predict the relative horizontal movement between the tube and side plate in Figure 6-20. However, the difference between the recorded displacement and the predicted displacement is within the tolerance of
the LVDTs and photogrammetry surveys. Overall the FE model showed very good agreement in both the tension and compression regions.

![Graph comparing FE and experiment for LVDT 1: overall beam deflection](image)

**Figure 6-16: Comparison of FE and experiment for LVDT 1 – overall beam deflection**
Figure 6-17: Comparison of FE and experiment for LVDTs 3 and 4 – tube to side plate

Figure 6-18: Comparison of FE and experiment for LVDTs 5 and 6 – beam flange to tube top face
Figure 6-19: Comparison of FE and experiment for LVDTs 7 and 8 – beam web to tube side face

Figure 6-20: Comparison of FE and experiment for LVDT 9– beam to tube (horizontal)
6.4.2 **Sensitivity analysis**

The FE model has shown a very good correlation with experimental results and hence is reliable for use in sensitivity analyses. Six parameters were chosen as variables to be investigated in the sensitivity analyses presented in this section, namely: (i) the coefficient of friction, \( \mu \) between surfaces, (ii) bolt size, (iii) side plate thickness, (iv) tube thickness, (v) channel thickness, and (vi) initial position of bolts with respect to bolt holes. It is the sensitivity of the initial connection stiffness to variation of these parameters that is of interest here.

6.4.2.1 **Coefficient of friction, \( \mu \)**

Using the FE model described earlier, the coefficient of friction, \( \mu \), between all contact surfaces in the model i.e. channel to beam flange, channel legs to side plates, side plates to SHS side walls and between bolt surfaces was changed from 0.15 to 0.2 and then 0.25. These values represent various levels of surface treatments ranging from painted to black steel. Increasing the slip coefficient increases the slip load dramatically and initial stiffness is also higher as shown in Figure 6-21. The tension load in the bolts was kept the same at 95kN. The slip load is directly proportional to \( \mu \). The friction coefficient, \( \mu \), between contact surfaces can be increased by removing the paint on the tube surface with hand wire brushing or light abrasive brushing. This will give a black steel to black steel contact surface which has an indicative slip factor of up to 0.30 as designated in AS4100 (Standards Australia, 1998). Without any treatments, the contact surfaces would be painted to black steel.
6.4.2.2 Bolt size

All the bolts in the FE model were changed from M16 to M20, keeping the same friction coefficient, $\mu$, at 0.15. A pretension load of 145kN was applied to the M20 bolts. By using larger size bolts, the initial stiffness and slip load were improved dramatically as shown in Figure 6-22. The initial stiffness increased by approximately 35% while the slip load increased by approximately 60%. Increasing bolt size is a simple and efficient way to improve the performance of the connection. Similar results could be achieved by increasing the number of bolts.

![Figure 6-21: Sensitivity analysis – varying coefficient of friction, $\mu$](image)
6.4.2.3 Side plate thickness

A plate thickness of 12mm was used in the test to ensure that the plates were stronger than the bolts such that the performance of the connection was not affected by the side plates. However, a plate of 10mm thick is more commonly used in industry practice. As part of the sensitivity analysis, the plate thickness in the FE model was changed from 12mm to 10mm while other parameters such as bolt size, tube thickness and $\mu$ remained the same. Figure 6-23 shows that there is little difference between the results for the two different plate thicknesses. It is recommended that plate thickness of not less than 10mm be used for this connection in order to ensure a low level of elastic deformation of the plates.

Figure 6-22: Sensitivity analysis – vary bolt size

$S_{j,\text{rigid (braced)}} = 16850 \text{ kNm/rad}$
6.4.2.4 Tube thickness

The 6mm thick 150SHS in the FE model was replaced firstly by 5mm and then 9mm wall thicknesses to investigate the effect of tube thickness on the stiffness of the connection. These thicknesses (5 mm, 6 mm and 9 mm) represent the existing sizes within the commercially available columns of this 150 mm square size. Figure 6-24 shows very similar behaviour for all three tube thicknesses indicating that for the 150 SHS, thickness of the tube does not affect the initial connection stiffness.

Figure 6-23: Sensitivity analysis – vary side plate thickness

![Rotation vs Moment Graph](image-url)
6.4.2.5 Channel thickness

The channel component in the connection was produced by splitting a segment of SHS tube used for the column into halves. The channel thickness can be varied by using a different thickness from the 150×150 SHS range. As part of the sensitivity analysis, the channel sections in the FE model were replaced with 9mm thick sections. Figure 6-25 shows that there is a 25% increase in initial connection stiffness (taken as the secant value at a moment of 20kNm) when the channel thickness is increased from 6mm (same thickness as the SHS column used in the test) to 9mm (the thickest commercially available 150 SHS).
6.4.2.6 Initial position of bolts with respect to bolt holes

The initial position of the bolts with respect to bolt holes was varied in the FE model. In the initial (default) model (Section 6.4.1); all bolts were centrally located in the bolt holes. During erection on site, the position of these bolts may vary from being centrally located in the bolt holes to bearing against the perimeter of the bolt holes. As part of the sensitivity analyses, the location of the bolts were adjusted such that these were in their likely positions during setup of the testing due to gravity action on the bolts as shown in Figure 6-26. Figure 6-27 shows the comparison of moment rotation curve between the default model with bolts centrally located and the offset bolt positions. It can be seen that the initial stiffness of the connection is only minimally influenced by the relative bolt positions with an approximately 8% increase in the secant stiffness at a moment of 20kNm for the offset condition. For higher moments the increase in stiffness would be more significant since the total amount of slip would be reduced for the offset condition.
Figure 6-26: Sensitivity analysis – vary position of bolts for top and bottom connections

Figure 6-27: Comparison of moment rotation curves when bolts were offsetted from center of bolt holes
6.5 Design models

In this section, a simplified design model for the channel side plate connection is developed to predict the stiffness and strength. A summary for the overall design of the channel side plate connection is given in Figure 6-28.

6.5.1 Strength

Possible failure modes for the channel side plate connection are bolts in shear, ply in bearing and tearing, plates in tension and compression and channel in shear. Design checks for these standard failure modes are well codified. Detailed procedures to design the channel side plate connection using Ajax ONESIDE blind bolts are outlined in Appendix B.

6.5.2 Serviceability

Under the serviceability condition the connection should be designed so that there is no slip of bolts. The slip load is based on the product of the pretension load in the bolts and the coefficient of friction, $\mu$, between connecting surfaces. For this type of connection, a large amount of slip will occur after the slip load is reached due to the contribution to slip from three locations: (i) bolts between beam flanges and channels, (ii) channel legs and side plates and (iii) side plates and tube side walls. In a design serviceability situation, the serviceability moment applied at the connection would be required to be less than the slip moment. The design criterion used to decide on the required minimum value of the slip moment would need to be assessed on a case-by-case basis.
Blind Bolted Connections for Steel Hollow Section Columns

Figure 6-28: Summary of channel side plate connection design

Step 1: Determine design actions on connection (M*, T* & V*) and determine design forces for each channel face, F* (as per Eq. A-1)

Step 2: Check for channel shear capacity to transmit F* (Eq. B-1)

Step 3: Determine minimum number of standard bolts, Nsb to transfer shear force in the channel face, check for bearing of ply (as per Eqs. A-7 and A-10)

Step 4: Determine minimum number of blind bolts, Nbb required to carry tension force from beam flange to column side walls, check for bearing of ply (as per Eqs. A-7 and A-10)

Step 5: Check side plates in tension and shear (as per Eqs. A-4 and B-2)

where $K_{\text{compression}} \approx K_{\text{tension}}$

$$\frac{1}{K_{\text{tension}}} = \frac{1}{K_{bc}} + \frac{1}{K_{cp}} + \frac{1}{K_{pt}} + \frac{1}{K_{\text{plate}}} + \frac{1}{K_{\text{channel}}} + \frac{1}{K_{\text{tube}}}$$

Refer to Eqs. 6-2 & 6-3 for $K_{bc}$, $K_{cp}$, $K_{pt}$, $K_{\text{plate}}$, $K_{\text{channel}}$ and $K_{\text{tube}}$

Hence, overall connection stiffness, $S_{j,ini}$:

$$S_{j,ini} = \frac{M}{\theta} = \frac{z^2}{\frac{1}{K_{\text{tension}}} + \frac{1}{K_{\text{comp}}} + \frac{2}{K_{\text{tension}}}} \approx \frac{z^2}{\frac{2}{K_{\text{tension}}}}$$
The initial stiffness of the blind bolted channel side plate connection can be predicted using the component approach proposed by Eurocode 3 (2005). From the experimental and FE observations the tension and compression regions of the connection can be assumed to behave in the same manner under applied loads. Hence, only one of the two regions is considered in detail. Each of the two regions of the connection is divided into an equivalent component model with six spring elements in series representing the following stiﬁnesses: (i) clamping of bolts between the beam and channels \(K_{bc}\), (ii) clamping of bolts between channel and side plates \(K_{cp}\), (iii) clamping of bolts between side plates and tube side walls \(K_{pt}\), (iv) plate extension \(K_{plate}\), (v) channel in shear \(K_{channel}\) and (vi) tube side wall in shear \(K_{tube}\).

The overall stiffness of the connection in the tension region, \(K_{tension}\), can be expressed as:

\[
\frac{1}{K_{tension}} = \frac{1}{K_{bc}} + \frac{1}{K_{cp}} + \frac{1}{K_{pt}} + \frac{1}{K_{plate}} + \frac{1}{K_{channel}} + \frac{1}{K_{tube}}
\]  

(6-1)

Each of these components is discussed in the following subsections.

6.5.2.1 Clamping stiffness for bolts \((K_{bc}, K_{cp}, K_{pt})\)

While fully tensioned bolts do not allow slip up to the load required to overcome the frictional force produced by clamping action across mating surfaces, such connections are not infinitely rigid. The linear springs with stiﬁnesses \(K_{bc}\), \(K_{cp}\) and \(K_{pt}\) represent the flexibilities at the three locations where tensioned bolts are employed in the connection.

In order to determine the clamping stiffness of the pretensioned M16 bolts used in the test, a total of 12 small-scale slip tests were conducted. The test setup is shown in Figure 6-29. Cutoffs from the 150×6 SHS tube were clamped to a 12mm thick middle plate. An LVDT was mounted on the middle plate, measuring the relative
movement between the middle plate and the SHS cutoffs. The average friction coefficient of the 12 specimens was found to be 0.18 with standard deviation of ±0.01. Friction coefficient, \( \mu \) was back calculated using the formula from Clause 9.3.3 AS 4100 (Standards Australia, 1998).

\[
V_{sf} = \mu k_h n_{ei} N_{ti}
\]

where

\( \mu \) = slip factor

\( k_h \) = hole factor

\( n_{ei} \) = No. of effective interfaces

\( N_{ti} \) = bolt tension at installation

The load at which the connection slipped, \( V_{sf} \) was recorded from the test. \( N_{ti} \) is the applied pretension in the bolt, \( k_h = 1 \) from AS 4100, \( n_{ei} = 2 \) for double interfaces.

The clamping stiffness was calculated from the recorded LVDT readings of relative movements between the middle plate and the SHS cutoffs. Given that these relative movements were very small, i.e. in the order of between 20 and 100 microns, the precision of the readings was limited by the accuracy of the LVDT (+ or – 30 microns). Hence the experimentally determined clamping stiffness varied from between 200kN/mm to 900kN/mm.
A simple FE model (shown in Figure 6-30) was created to determine the clamping stiffness of a pretensioned M16 bolt (pretension load is 95kN). The middle plate was subjected to a tension load. Relative movement between the plates before slip occurred was used to determine the clamping stiffness of the bolt. The FE model produced a clamping stiffness of 550kN/mm per M16 bolt per shear plane for a friction coefficient, $\mu$, of 0.15 which is approximately the average value obtained from the experimental results. Hence this value is adopted in the component model calculation.

In the side plate connection, interfaces between beam flange and channel face, channel leg and plate, and plate and tube side wall have four M16 bolts connecting the surfaces. Hence, the clamping stiffness at each of these three locations, $K_{bc}$, $K_{cp}$, and $K_{pt}$, is given by $4 \times 550 = 2200\text{kN/mm}$. 

Figure 6-29: Slip test setup with one plate sandwiched between two channels
6.5.2.2 Extension of plates, $K_{\text{plate}}$

Extension of side plates, $K_{\text{plate}}$ is calculated from Equation 6-2:

$$K_{\text{plate}} = \frac{EA_{\text{plate}}}{L_{\text{plate}}}$$  \hspace{1cm} (6-2)

where $E = \text{Young’s Modulus for steel, 200,000MPa}$

$A_{\text{plate}} = \text{cross sectional area of plate}$

$L_{\text{plate}} = \text{length of plate undergoing extension}$

For the test specimen,

$$K_{\text{plate}} = \frac{200000 \times 90 \times 12 \times 2 \times 10^{-3}}{170} = 2541 kN / mm$$  \hspace{1cm} (6-2a)

6.5.2.3 Channel in shear, $K_{\text{channel}}$

The channel section connecting beam flange to side plate is subjected to shear action between the channel web and flange. Shear stiffness of a section is calculated from Equation 6-3 (Timoshenko & Gere, 1972):

Shear stiffness $K_v = \frac{GA_v}{\alpha_{\alpha_v}}$  \hspace{1cm} (6-3)
where \( G \) = shear modulus for steel 80000MPa

\[ A_v = \text{cross sectional area in shear} \]

\[ \alpha_s = \text{shear coefficient} \]

\[ x = \text{deformed length in shear} \]

Stiffness of channel in shear, \( K_{\text{channel}} \) was calculated based on Equation 6-3 with \( \alpha_s \) of 6/5 for rectangular cross section (Timoshenko & Gere, 1972). Hence, for the test specimen:

\[
K_{\text{channel}} = \frac{80000 \times 150 \times 6 \times 2 \times 10^{-3}}{6.5 \times 79.3} = 1513kN/mm 
\]

(6-3a)

6.5.2.4 Tube in shear, \( K_{\text{tube}} \)

The SHS column is also subjected to shear action from the load transfer in the connection (panel zone deformation). Stiffness of the tube in shear is calculated from Equation 6-3 with \( \alpha_s \) of 1.35 which was calibrated from FE modelling. Therefore, for the test specimen:

\[
K_{\text{tube}} = \frac{80000 \times 3330 \times 10^{-3}}{1.35 \times 178} = 1100kN/mm
\]

(6-3b)

6.5.2.5 Assembly

The connection stiffness in the tension region (\( K_{\text{tension}} \)) is estimated to be 300kN/mm based on Equation 6-1. With the tension and compression stiffness of the connection being the same, overall connection stiffness can be calculated from Equation 6-4:

\[
S_{j,\text{int}} = \frac{M}{\theta} = \frac{z^2}{\frac{1}{K_{\text{tension}}} + \frac{1}{K_{\text{compression}}}} = \frac{z^2}{2/\frac{1}{K_{\text{tension}}}} 
\]

(6-4)

where \( z \) is the lever arm of the connection \( \approx 300mm \). This gives an overall calculated connection stiffness of 13525kNm/rad as compared to 17000kNm/rad
from the experimental results. This is a conservative value being 20% lower than
the actual stiffness of the specimen and 13% lower than the prediction from FE
model of 15500kNm/rad.

A comparison is made in Table 6-2 between the predictions from component
model and FE analyses of connection stiffness in the tension region for the various
cases covered by the sensitivity analysis in Section 6.4.2. The component model
underpredicts the FE analysis by approximately 10% when the channel thickness is
6mm and underpredicts by 25% when the channel thickness is increased to 9mm.

Table 6-2: Comparison of component model to FE prediction of tension region stiffness (150×150
SHS)

<table>
<thead>
<tr>
<th>Model</th>
<th>Tube thk (mm)</th>
<th>Side plate thk (mm)</th>
<th>Channel thk (mm)</th>
<th>$K_{tension}$ (kN/mm)</th>
<th>$K_{component}$</th>
<th>$K_{component}/K_{FE}$</th>
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<td>12</td>
<td>9</td>
<td>525</td>
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</tr>
</tbody>
</table>

Table 6-3 shows a comparison between the stiffnesses of the tension and
compression regions for the channel side plate connection. The stiffness of the
compression region is approximately 7 to 10% higher than the tension region and
hence for simplicity, it can be assumed to have the same stiffness as the tension
region in the assembly of the component model. The component model despite
its simplicity is able to predict the connection stiffness quite well.

Table 6-3: Comparison of stiffness for tension and compression regions (150×150 SHS)

<table>
<thead>
<tr>
<th>Model</th>
<th>Tube thk (mm)</th>
<th>Side plate thk (mm)</th>
<th>Channel thk (mm)</th>
<th>$K_{tension}$</th>
<th>$K_{compression}$</th>
<th>$K_{compression}/K_{tension}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6</td>
<td>12</td>
<td>6</td>
<td>404</td>
<td>439</td>
<td>1.09</td>
</tr>
<tr>
<td>2</td>
<td>5</td>
<td>12</td>
<td>6</td>
<td>406</td>
<td>434</td>
<td>1.07</td>
</tr>
<tr>
<td>3</td>
<td>9</td>
<td>12</td>
<td>6</td>
<td>400</td>
<td>430</td>
<td>1.08</td>
</tr>
<tr>
<td>4</td>
<td>6</td>
<td>10</td>
<td>6</td>
<td>390</td>
<td>428</td>
<td>1.10</td>
</tr>
<tr>
<td>5</td>
<td>6</td>
<td>12</td>
<td>9</td>
<td>525</td>
<td>566</td>
<td>1.08</td>
</tr>
</tbody>
</table>
6.6 Summary and conclusions

A new bolted moment resisting connection to hollow section columns utilising blind bolts and side plates was developed and tested. A detailed FE model and a design model have been developed for this connection. The findings from the test and FE analysis may be summarised as follows:

1) The results show that the channel side plate blind bolted connection has the potential to be classified as a rigid connection for a braced frame system according to the Eurocode 3 specifications. The stiffness of the connection is not compromised by the flexible hollow section column face deformation normally observed in a typical face connection.

2) Comparing the stiffness of the channel side plate connection tested in this chapter with a 10mm thick T-stub connection for the same size tube and beam, the channel side plate connection is approximately five times stiffer than the T-stub connection. This highlights the benefit of connecting to the side faces of the SHS column as compared to the conventional front face connection.

3) A 3-D non-linear finite element model was developed for the channel side plate connection. The FE deformations of the various components were compared with the detailed experimental results particularly from the high resolution photogrammetry measurements. It can be concluded that the FE model can predict the behaviour of the new connection with good level of accuracy.

4) Based on the sensitivity analyses performed using finite element modelling, it was found that a higher coefficient of friction, \( \mu \), between contact surfaces leads to a higher slip load and initial stiffness. Increasing
the bolt sizes also has a pronounced effect on the initial stiffness and slip load of the connection. A thicker channel also contributes to a higher initial connection stiffness. On the other hand, plate and tube thicknesses as well as initial position of bolts have minimal effects on the initial connection stiffness.

5) Friction tests were conducted for cutoffs from 150×150×6SHS tube clamped to a 12mm thick plate. The average friction coefficient was found to be 0.18.

6) A component model is proposed in this chapter to predict the initial stiffness of the channel side plate connection. The component model considers clamping of bolts, plate extension, channel in shear and tube side wall in shear. Despite its simplicity, the component model is able to predict the stiffness of the connection with reasonable accuracy.

7) The channel side plate connection is versatile and is easy to assemble on site. However it requires all blind bolts to be fully pretensioned to achieve the required stiffness. There is a need to consider an alternative connection which requires less reliance on pretensioning of blind bolts.
Chapter 7

Extended T-stub connection with back face support

7.1 Introduction

An alternative connection to the channel side plate connection discussed in Chapter 6 is proposed in this chapter which aims to achieve comparable initial stiffness as the channel side plate connection with less reliance on pretensioning of blind bolts.

Results of an experimental and analytical study are presented for a new bolted moment connection between unfilled hollow section columns and open section beams, referred to herein as the extended T-stub connection with back face support (refer Figure 7-1). The connection is comprised of T-stubs connecting the top and bottom flanges of the beam to the face of the column and channels connecting the T-stubs to a backplate at the back face of the hollow section column or to a T-stub at the back face if a beam is also being connected there. The addition of the channels to a standard T-stub connection helps to distribute the beam flange tension load to the back face of the hollow section column. This
reduces the demand on the flexible column face hence increasing the stiffness of the connection when compared with the standard T-stub connection.

7.2 Experimental program

7.2.1 Test specimen

A full scale beam-to-column extended T-stub connection with back face support was developed for testing. Details of the specimen are shown in Figure 7-1. The specimen comprised a 310UB32 universal beam (approximately 310mm deep, 32kg/m section) connected to a 150×150×6mm SHS column which are the same member sizes adopted in the previous test regime. The channels and T-stubs used in the connection can be made by cutting readily available commercial sections. The channel sections used in this test were made by cutting into halves the thickest commercially available 150×150 SHS, i.e. 9mm. The T-stubs were fabricated from full penetration butt welding of plates. For mass production, the T-stubs could be cut from a 1000WB215 for the same thicknesses of the endplate and T-stem as were used in this test.

The beam flanges were bolted to the stem of the T-stub with six standard M16 Grade 8.8 bolts at the top and bottom respectively. The centre section of the T-stubs was then blind bolted to the tube face with Grade 8.8 M16 ONESIDE blind bolts while the outer section was bolted to the channel flanges with standard M16 structural bolts. The opposite flanges of the channels were then bolted to a 20mm thick backplate to transfer forces to the back face of the column. None of the blind bolts was pretensioned but were simply tightened to the “snug tight” condition in accordance with AS 4100 (Standards Australia, 1998). All other standard structural bolts were pretensioned with a torque wrench.
7.2.2 Experimental setup and instrumentation

A hydraulic actuator was used to apply horizontal load at the cantilever end of the beam as shown in Figure 7-1. The SHS column was restrained at both ends and tied down to the laboratory strong floor. Displacement transducers (LVDTs) were used to measure relative movements between the T-stubs, channels, beam and the tube. An illustration of the instrumentation used to monitor the behaviour of specimen is shown in Figure 7-2. Digital photogrammetry was also employed in this test. Photogrammetry targets were placed on the channels, T-stubs, beam and SHS tube to obtain the overall deformation profile.
7.3 Test results

7.3.1 General behaviour and failure load

Loading was gradually applied to the test specimen at the end of the cantilever beam up to failure. As the load was increased, the beam started to rotate in the direction of the applied load. Slip started to occur at an applied load of approximately 40kN between the beam flange and T-stem as indicated by the onset of non-linearity at this load level in Figure 7-3 to Figure 7-6. It is the stiffness of the connection up to the slip load that is of most interest in this research. At an applied load of 80kN, the 9mm thick channels yielded and the top flange of the channels (connected to the T-stub) moved in an outward direction, away from the column. As the channels in the tension region of the connection opened up, the SHS column face was also pulled outwards by the T-stub which was connecting the channels and column face together. Due to the high stiffness of the...
T-stub tying the tube face and channels together, the SHS column face deformation was constrained to be the same as the deformation of the channel top flanges. Figure 7-7 shows the deformation of the channels and tube face in the tension region at failure. Significant rotation was observed at the corners of the channels indicating that plastic deformation had occurred.

At an applied load of 240kN, failure of the standard bolts on the beam flange was imminent and the test was stopped at this point. This load corresponds to a shear force of 115kN in each bolt on the beam flange which is already slightly higher than the code-estimated shear capacity for 8.8 M16 bolts. The deformed shape of the specimen at an applied load of 240kN is shown in Figure 7-8a and b.

**7.3.2 Overall beam end deflection – LVDT 1**

Figure 7-3 shows the overall beam end deflection where the connection displays high initial stiffness up to a load of 40kN at which slip commenced. The additional flexibility in post-slip stiffness compared with the initial stiffness is attributed to the continued slippage of the bolts while the connection was loaded and eventually to yielding of the channels and beam. Also shown in Figure 7-3 are the results from the photogrammetry surveys which match very well with the LVDT readings.
Figure 7-3: Load vs. displacement for LVDT 1 – overall beam flange displacement, $\Delta_x$ (refer to Figure 7-2 for location of LVDT)

Figure 7-4: Load vs. displacement for LVDTs 2 and 3 – beam flange to T-stem, $\Delta_y$ (refer to Figure 7-2 for locations of LVDTs)
Figure 7-5: Load vs. displacement for LVDTs 4 and 5 – T-stem to endplate, $\Delta_Y$ (refer to Figure 7-2 for locations of LVDTs)

Figure 7-6: Load vs. displacement for LVDTs 6 and 7 – beam web to endplate, $\Delta_Y$ (refer to Figure 7-2 for locations of LVDTs)
Figure 7.7: Deformation of tube face and channels at failure

a) Overall deformed shape at termination of test
7.3.3  Relative movement between beam flange and T-stem – LVDTs 2 and 3

A similar trend was observed for the relative movement between the beam flanges and T-stems in both the tension and compression regions in Figure 7-4. Bolts connecting the beam and T-stem started to slip at a load of 40kN. The compression flange appeared to experience a lesser amount of slip, most likely due a different initial position of the bolts in the bolt holes.

7.3.4  Relative movement between T-stem and endplate – LVDTs 4 and 5

Relative movement between the T-stem and endplate in the tension and compression regions appeared to be quite symmetrical as shown in Figure 7-5.
7.3.5  **Relative movement between beam web and endplate – LVDTs 6 and 7**

Figure 7-6 shows the relative movement between the beam web and the endplate in both the compression and tension regions. Similar initial behaviour in both regions was observed until slip started to occur. In the tension region, slip continued to occur from an applied load of 40kN to 100kN before stiffness increased again as observed in the earlier graphs. The compression region appeared to behave quite differently to the tension region.

LVDT 6 was mounted close to the tension flange hence it recorded a positive displacement indicating increase in gap between the beam and the endplate while LVDT 7 was located close to the compression flange and hence when slip commenced, it showed negative displacement indicating that this end of the beam moved closer to the endplate as shown in Figure 7-6.

With further loading beyond the slip load, the compression flange did not move further down as the T-stub was in direct bearing on the column face. Therefore the compression end of the connection was very stiff up to applied load of 150kN. At applied loads above 150kN, the hard point of rotation of the connection was at the intersection between the T-stub stem and endplate. Therefore LVDT 7 recorded tension displacements as opposed to compression due to the rotation of the beam about this hard point.

7.3.6  **Relative movement between channel flanges – LVDTs 8 to 10**

Figure 7-9 shows the relative moment between channel flanges in the tension region for both sides of the tube from LVDTs 8 and 9. The channels on both sides behaved symmetrically until slip occurred. At higher loads above 200kN, as the
beam started to yield and underwent lateral torsional buckling, the loading on both channels would have differed slightly. Relative movement between the channel flanges in the compression region (LVDT 10) as shown in Figure 7-10 is very minimal as the load was transferred mainly by bearing on the tube face rather than going through the channels.

Figure 7-9: Load vs. displacement for LVDTs 8 and 9 – channels in tension, $\Delta_Y$ (refer to Figure 7-2 for locations of LVDTs)
7.3.7  **Moment rotation response**

The load-displacement curve from the experimental results was converted to a moment-rotation curve for the beam used in the experiment (310UB32) as shown in Figure 7-11.

Assuming a 6m beam length which is common for low rise residential structures, the classification limits based on the properties of 310UB32 beam are plotted on Figure 7-11. It can be seen that the initial stiffness of the connection was found to be 17000kNm/rad. This is considered as rigid for braced frames based on the Eurocode 3 classifications. This condition held up to a moment of 40kNm at which slip started to occur and the connection became more flexible when subjected to further moment.
A brief comparison can be made with the stiffness of the blind bolted connections tested in the earlier experiment regime, the ordinary T-stub connection (Chapter 4) and channel side plate connection (Chapter 6). For the same size SHS column and beam, the stiffness of the 10mm thick T-stub connection was estimated to be 3500kNm/rad while the channel side plate connection had a stiffness of 17000kNm/rad. The extended T-stub connection with back face support is approximately five times stiffer than the ordinary 10mm thick T-stub connection and has similar stiffness to the channel side plate connection.

Finite element analysis and sensitivity studies were carried as outlined in Section 7.4 to extend the experimental results and specifically investigate parameters that may affect the initial stiffness and slip load of the extended T-stub connection with back face support.

![Diagram](image)

**Figure 7-11: Moment rotation response of extended T-stub connection**
7.4 Finite element analysis

A three-dimensional finite element (FE) model was created using the general purpose software ANSYS to represent the tested connection. The FE model takes into account material and geometric non-linearities and complex contact interactions between the various elements. Surface to surface contact elements with friction coefficient, $\mu$, of 0.15 (which is the commonly adopted $\mu$ for painted surfaces) were employed in the FE model. $\mu$ was also validated by separate friction tests discussed in Chapter 6.

Taking advantage of symmetric conditions along the longitudinal plane, one half of the test specimen was modelled. A full FE model of the channel and T-stub bearing connection is shown in Figure 7-12 for ease of visualisation. The load was applied at the shear centre of the beam web and a line of supports was provided at the top and bottom faces of the tube in both the tension and compression regions to simulate support conditions during the experiment.

The FE model was used to predict the initial stiffness of the connection, slip between the T-stem and beam flange and deformation of all elements. The material properties of the SHS column, beam, channels, T-stubs and bolts were described by bilinear stress-strain curves based on the von Mises yield criteria with rate independent isotropic work hardening. The yield stress and tangent modulus of the various elements are summarised in Table 7-1, their basic steel properties were obtained through tensile tests. The various components of the blind bolts were also modelled. The sleeves’ inner diameter was 16.5mm and outer diameter was 22.5mm leaving a gap of 0.5mm to the 16mm diameter bolt shank and 1.5mm to the 24mm diameter bolt holes.
### Table 7-1: Material properties for bilinear model

<table>
<thead>
<tr>
<th>Item</th>
<th>Yield Stress (MPa)</th>
<th>Elastic modulus, $E_o$ (MPa)</th>
<th>Tangent modulus, $E_{tgt}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SHS</td>
<td>350</td>
<td>200000</td>
<td>$0.01E_o = 2000$</td>
</tr>
<tr>
<td>Channels</td>
<td>350</td>
<td>200000</td>
<td>$0.01E_o = 2000$</td>
</tr>
<tr>
<td>Beam</td>
<td>300</td>
<td>200000</td>
<td>$0.01E_o = 2000$</td>
</tr>
<tr>
<td>T-stub</td>
<td>300</td>
<td>200000</td>
<td>$0.01E_o = 2000$</td>
</tr>
<tr>
<td>Backplate</td>
<td>300</td>
<td>200000</td>
<td>$0.01E_o = 2000$</td>
</tr>
<tr>
<td>Bolts</td>
<td>640</td>
<td>200000</td>
<td>$0.01E_o = 2000$</td>
</tr>
</tbody>
</table>

**Figure 7-12: FE model of extended T-stub connection**

#### 7.4.1 Comparison to test results

The FE model was extensively validated using the experimental results as obtained through the LVDTs and photogrammetry surveys.

The comparison for beam end deflection (LVDT 1) is shown in Figure 7-13. The FE model is able to predict the overall deflection very well. Figure 7-14 shows a good match between the FE and LVDT records for the movement between the beam flange and the T-stem before slip occurred in the bolts. The FE model gives very accurate prediction of the tension region and also of the compression region up to
an applied load of 40kN. The FE model was used to estimate the stiffness prior to slip and was not continued beyond this level.

The tension region appears to have a higher slip load compared to the FE model. Similar trends are also observed in Figure 7-15 to Figure 7-17 suggesting that the surface contacts in the tension region may have a slightly higher $\mu$. The slip load is largely dependent on the value of $\mu$ between contacting surfaces, pretension load in the bolts and the location of the bolts within the bolt holes. Average values of these three parameters have been used in the FE model. Adopting a higher $\mu$ of 0.2 in the FE model will give an increase in the slip load from 60kN to 75kN, which matches the displacements in the tension region very well (LVDTs 2 and 6). However this will overpredict the results in the compression region (LVDT 3). The discrepancies between the FE and experimental results are due to the variation in tensioning of bolts and in the location of bolts within the bolt holes during the experimental setup of the connection in the tension and compression regions.

![Figure 7-13: Comparison of FE and experiment for LVDT 1 – overall beam flange displacement](image-url)
Figure 7-14: Comparison of FE and experiment for LVDTs 2 and 3 – beam flange and T-stem

Figure 7-15: Comparison of FE and experiment for LVDT 6– beam web to endplate (tension)
Blind Bolted Connections for Steel Hollow Section Columns

Figure 7-16: Comparison of FE and experiment for LVDT 7 – beam web to endplate (compression)

Figure 7-17: Comparison of FE and experiment for LVDTs 8 and 9 – channels in tension
7.4.2 **Sensitivity analysis**

The FE model has shown a good correlation with experimental results and hence is reliable for use in a sensitivity analysis. As part of a brief sensitivity analysis related to the initial connection stiffness, three parameters were considered namely: (i) channel thickness, (ii) T-stub and backplate thicknesses and (iii) tube thickness.

7.4.2.1 Channel thickness

Using the FE model described earlier, the 9mm channel sections in the model were replaced with 6mm thick channels (which can be easily fabricated from cutting the column section, 150×6 SHS into two halves). The initial stiffness of the connection was reduced significantly by an amount of 20%, as observed in Figure 7-18. The corners of the thinner channel section are less stiff and open up more easily when the channel flanges are pulled outwards by the T-stub. Therefore, the channel thickness adopted for this connection would preferably be greater than the column thickness.
7.4.2.2 T-stub and backplate thickness

The thicknesses of the endplate and stem which made up the T-stub, and the backplate, were varied as part of the sensitivity analysis. In the first model, the thickness of the endplate and backplate was changed from 20mm to 10mm, keeping the same thickness for the stem. Another model was also generated with the endplate, stem and backplate all having a reduced thickness of 10mm. Figure 7-19 shows the comparison between the initial (default) FE model and the modified T-stub thickness model. The thickness of the endplate and backplate is crucial for the stiffness of the connection, a 10mm endplate and backplate resulted in a 55% reduction of the initial connection stiffness. The thinner plates deformed easily in bending; thus accounting for this much reduced connection stiffness. In comparison, it can be seen that having a reduced stem thickness, from 16mm to 10mm, has little effect on the initial stiffness as these elements are loaded in tension rather than bending.
7.4.2.3 Tube thickness

The thickness of the tube was increased from 6mm in the default FE model to 9mm. Comparison of moment rotation curves for both scenarios is shown in Figure 7-20. Increasing the tube thickness from 6mm to 9mm resulted in 30% additional stiffness which is a significant increase. A thicker tube has more resistance to tube face deformation thereby resulting in an overall stiffer connection.
7.5 Design models

In this section, simplified design models for the extended T-stub connection with back face support to predict the strength and stiffness are discussed and summarised in Figure 7-21.

Figure 7-20: Sensitivity analysis – vary tube thickness
Step 1: Determine design actions on connection (M’, T’ & V’) and determine design forces for each T-stem, F’

Step 2: Check thickness of T-stem, t_{stem} required to transmit tension force in the T-stem

Step 3: Determine minimum number of standard bolts, N_{sb} to transfer shear force in the T-stem and to ensure a sufficiently high slip load

Step 4: Determine minimum number of blind bolts, N_{bb} required to carry tension force from beam flange to column face, check blind bolts for shear and combined actions. A two tiered approach: First tier assumes all the load being taken by the tube face. If inadequate, perform the second tier of analysis based on distributing the flange load between the tube face and back plate in accordance with their stiffnesses.

Step 5: Check for column face yielding and pull-out action of blind bolts from tube face

Step 6: Check for channels yielding based on simplified channel model in Figure 7-23

Step 7: Check for backplate in bending based on simplified backplate model in Figure 7-24

Component method based on Eurocode3

Refer to Eqs. 7-2 to 7-6 for K_{clamp}, K_{stem}, K_{endplate}, K_{channels}, K_{backplate}, and K_{tube} respectively

Hence, overall connection stiffness, S_{j,ini}:

\[ S_{j,ini} = \frac{M}{\theta} = \frac{z^2}{\left(\frac{1}{K_{tension}} + \frac{1}{K_{comp}} + 1.2\frac{1}{K_{tension}}\right)} \]

where \( K_{compression} \approx 5K_{tension} \)
7.5.1 **Strength**

There are a number of possible failure modes which need to be considered for the extended T-stub connection with back face support. These include:

(i) bolts in shear for those connecting the T-stem and beam flange,

(ii) bolts in combined tension and bending for those connecting the endplate to the column face, endplate to channels, and channels to backplate,

(iii) ply in bearing and tearing, and

(iv) bending failure of the endplate.

Design checks for these failure modes are well codified in AS4100 (Standards Australia, 1998) and design example of a bolted endplate connection is given in AISC Design of Structural Connections (Hogan & Thomas, 1994) and CIDECT Design Guide 9 (Kurobane et al., 2005), hence they are not covered in this chapter. For simplicity, the design procedures for a typical T-stub connection as outlined in Appendix A may be adopted for the extended T-stub connection with back face support.

A simplified design procedure for the extended T-stub connection is given below:

- Determine design actions on connection \((M^*, T^* & V^*)\) and determine design forces for each T-stem, \(F^* = \frac{M^*}{d_{bm}}\) where \(d_{bm}\) is the depth of beam.

- Check thickness of T-stem, \(t_{stem}\) required to transmit tension force in the T-stem.

- Determine minimum number of standard bolts, \(N_{sb}\) to transfer shear force in the T-stem and to ensure a sufficiently high slip load.

- Determine minimum number of blind bolts, \(N_{bb}\) required to carry tension force from beam flange to column face, check blind bolts for shear and combined actions. A two tiered approach is suggested. The first tier of analysis is based on the very conservative approach of all the load from the
beam flange being taken by the tube face. This is to be done with the minimum number of bolts necessary to perform the connection. If the face connection is found to be inadequate, then the second tier of analysis is to be performed. This would be based on distributing the flange load between the tube face and back plate in accordance with the stiffnesses of these two load paths (i.e. $K_{\text{tube}}$ and $K_{\text{backplate}} + K_{\text{channels}}$, refer Section 7.5.2).

- Check for column face yielding and pull-out action of blind bolts from tube face.
- Check for T-stub yielding and combined bolt or T-stub failure.
- Check for channels yielding based on simplified channel model in Figure 7-23.
- Check for backplate in bending based on simplified backplate model in Figure 7-24.

### 7.5.2 Serviceability

Under the serviceability condition, the connection can be designed to prevent slip of bolts in the beam connection to the T-stem based on the number of bolts, pretension load in the bolts and coefficient of friction, $\mu$ between connecting surfaces. In a design serviceability situation, the serviceability moment applied at the connection would be required to be less than the slip moment. The design criterion used to decide on the required level of the slip moment would need to be assessed on a case-by-case basis. For deformation sensitive structures, the designer might require that the ultimate design moment to be less than or equal to the slip moment.

As this connection is made up of various components interacting together with load sharing between the tube and channels, predicting the initial stiffness of this connection is a complex exercise. This section provides a procedure to estimate
the initial stiffness of the extended T-stub connection with back face support based
on the classical component method approach proposed by Eurocode 3 (European
Committee for Standardisation (CEN), 2005). More refined predictions can be
provided through design charts and tables generated from FE analyses.

The tension region of the connection is comprised of a combination of springs in
series and springs in parallel. Springs act in series when the components
experience the same force, these are (i) clamping stiffness of bolts for beam flange
and T-stem ($K_{\text{clamp}}$), (ii) T-stem extension ($K_{\text{stem}}$), and (iii) endplate in bending
($K_{\text{ep}}$). Springs act in parallel when the components undergo the same
displacement, and these are (iv) combined stiffness (in series) of channels opening
up ($K_{\text{channels}}$) and backplate in bending ($K_{\text{backplate}}$), and (v) tube face stiffness ($K_{\text{tube}}$).
The overall stiffness of the joint in tension, $K_{\text{tension}}$ can be expressed by Equation 7-1
and is illustrated in Figure 7-22.

$$K_{\text{tension}} = \frac{1}{\frac{1}{K_{\text{clamp}}} + \frac{1}{K_{\text{stem}}} + \frac{1}{K_{\text{ep}}} + 1} + K_{\text{tube}}$$

(7-1)

Each of these components is discussed below.

![Figure 7-22: Component model for tension region](image-url)
7.5.2.1  Clamping stiffness for bolts ($K_{\text{clamp}}$)

Although the bolts connecting the beam flanges to the T-stem were fully pretensioned, there was some flexibility in the clamped surfaces before slippage occurs between the mating surfaces. Based on 12 samples of friction tests conducted previously as shown in Figure 6-29, the average clamping stiffness of an M16 bolt per shear plane for a friction coefficient, $\mu$, of 0.15 is approximately 550kN/mm. There were six M16 bolts connecting the surface of the beam flange to the T-stem in the test specimen described in Section 7.2. Hence, the clamping stiffness $K_{\text{clamp}}$ is given by $6 \times 550 = 3300$ kN/mm.

7.5.2.2  Extension of T-stem ($K_{\text{stem}}$)

For the extension of T-stem plates, $K_{\text{stem}}$ is calculated from Equation 7-2:

$$K_{\text{stem}} = \frac{EA_{\text{stem}}}{L_{\text{stem}}} \quad (7-2)$$

where $E$ = Young’s Modulus for steel, 200 000 MPa
$A_{\text{stem}}$ = cross sectional area of stem
$L_{\text{stem}}$ = length of stem undergoing extension

For the test specimen, this would be:

$$K_{\text{stem}} = \frac{200000 \times 300 \times 16 \times 10^{-3}}{110} = 8700 \text{ kN/mm} \quad (7-2a)$$

7.5.2.3  Endplate in bending ($K_{\text{ep}}$)

Stiffness of the endplate is predicted based on simple beam theory as discussed in Chapter 4. A fixed end is assumed at the intersection of endplate and stem while a line load is assumed at the bolt locations. The stiffness of the endplate is based on a cantilever analogy and is given by Equation 7-3,

$$K_{\text{ep}} = 0.5E \frac{b_{\text{eff}} t_{\text{ep}}^3}{m^3} \quad (7-3)$$
where \( E \) = Young’s modulus for steel, 200 000MPa
\( b_{eff} \) = effective width of endplate (refer (Faella et al., 2000)), is taken as width of endplate for the configuration used in the experiment
\( t_{ep} \) = thickness of endplate
\( m \) = distance from intersection of stem and endplate to bolt location

For the test specimen, this would be:

\[
K_{ep} = \frac{0.5 \times 200000 \times 300 \times 20^3 \times 10^{-3}}{57^3} = 1296\text{kN/mm} \quad (7-3a)
\]

7.5.2.4 Channel opening up (\( K_{channels} \))

The channel sections transferring the load from the endplate to the back face of the tube in bearing is modelled as shown in Figure 7-23.
Stiffness of the channel section opening up is calculated using a virtual work approach: a unit horizontal load is applied at the location of the bolt force, \( P \) to obtain an expression for the horizontal displacement at the tip of the channel. The equation is then rearranged to obtain an expression for the stiffness of the channels as presented by Equation 7-4:

\[
K_{\text{channels}} = \frac{EI_{\text{channel}}}{\left(\frac{2}{3}a^3 + a^2b\right)} \times 2 \quad (7-4)
\]

where
- \( I_{\text{channel}} \) = second moment of area for channel plate section
- \( a \) = dimension of simplified channel flange, from edge of channel corner to centre of bolt
- \( b \) = dimension of simplified channel leg, edge to edge distance of channel corners

Hence, for the test specimen, this would be:

\[
K_{\text{channels}} = 2 \times \frac{200000 \times \left(266 \times 9^3 / 12 \right) \times 10^{-3}}{\frac{2}{3} \times 19.5^3 + 19.5^2 \times 105} = 144kN/mm \quad (7-4a)
\]

7.5.2.5 Tube face deformation \((K_{\text{tube}})\)

The flexible column face bending stiffness is predicted based on simplified finite element modelling performed by Mourad (1994). The stiffness of the column face in bending, \( K_{\text{tube}} \) is given by Equation 7-5,

\[
K_{\text{tube}} = \frac{Et_c^3}{12(1-\nu^2)R^* \gamma_s (b_c - 2t_c)^2} \quad (7-5)
\]

where
- \( E \) = Young’s modulus for steel, 200 000MPa
- \( t_c \) = thickness of column
- \( b_c \) = width of column
- \( \nu \) = Poisson’s ratio for steel, 0.3
- \( \gamma_s \) = deflection coefficient (read from charts)
- \( R^* \) = reduction factor due to corner restraints (read from charts)
For the test specimen presented in this chapter, $K_{\text{tube}} = 81kN/mm$ from Equation 7-5.

### 7.5.2.6 Backplate in bending ($K_{\text{backplate}}$)

The backplate connecting the back face of the tube and channel flanges can be designed as a cantilever subjected to a point load from the channel bolt force as shown in Figure 7-24. The length of the cantilever, $l'$ is taken from the corner of the tube section to the bolt centre. Stiffness of the backplate, $K_{\text{backplate}}$ can be calculated using Equation 7-6,

$$K_{\text{backplate}} = \frac{3EI_{\text{backplate}}}{l'^3}$$  \hspace{1cm} (7-6)

where $I_{\text{backplate}} =$ second moment of area of backplate $l'$ = corner of tube to centre of bolt on channel

Therefore, for the test specimen, this would be:

$$K_{\text{backplate}} = 2 \times \frac{3 \times 200000 \times \left(\frac{266 \times 20^3}{12}\right) \times 10^{-3}}{\frac{57^3}{57^3}} = 1149kN/mm$$  \hspace{1cm} (7-6a)
7.5.2.7 Assembly

For the test specimen, the connection stiffness in the tension region calculated from Equation 7-1 is estimated to be $K_{\text{tension}} = 167\text{kN/mm}$. From the earlier investigation of a typical T-stub connection in Chapter 4, the compression region can be conservatively assumed to be 5 times stiffer than the tension region of the connection. Hence, the overall connection stiffness can be calculated from Equation 7-7:

$$S_{j,\text{int}} = \frac{M}{\theta} = \frac{z^2}{\frac{1}{K_{\text{tension}}} + \frac{1}{K_{\text{compression}}}} = \frac{z^2}{1.2\times K_{\text{tension}}}$$  \hspace{1cm} (7-7)

where $z$ is the lever arm of the connection $\approx 316\text{mm}$ for 16mm thick stem and 300 deep beam. This gives an overall calculated connection stiffness of $13900\text{kNm/rad}$ as compared to $17000\text{kNm/rad}$ from the experiment results and $16000\text{kNm/rad}$ from FE model. The FE model underpredicts the connection stiffness by 8%; this is due to conservatism in adopting the material properties, coefficient of friction and contact properties in the FE model. Stiffness of the connection needs to be taken into account with the respective stiffness of beams and columns in the overall frame stiffness.
The component model was developed to be simple enough to be used for design yet conservative in its estimate. The model contains simplifications of complex interactions in the connection. If more accurate results are required, FE modelling can be carried out to obtain the connection stiffness.

To validate the component model further, a comparison of the component model and FE predictions for the various cases discussed in Section 7.4.2 is made in Table 7-2. The component model consistently produces conservative predictions and underestimates the tension region stiffness compared with FE model.

<table>
<thead>
<tr>
<th>Model</th>
<th>Endplate thk (mm)</th>
<th>Backplate thk (mm)</th>
<th>Stem thk (mm)</th>
<th>Channel thk (mm)</th>
<th>( K_{\text{tension}} ) (kN/mm)</th>
<th>( K_{\text{FE}} )</th>
<th>( K_{\text{component}} )</th>
<th>( K_{\text{component}}/K_{\text{FE}} )</th>
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</thead>
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<td>128</td>
<td>98</td>
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</tbody>
</table>

Table 7-3 shows a comparison between the stiffness of the connection in the tension and in the compression regions using the FE prediction for the various cases discussed in Table 7-2. The compression region of the extended T-stub connection with back support is generally five to seven times stiffer than the tension region for a 150×6SHS column. Therefore the assumption made in the component model that \( K_{\text{compression}} = 5K_{\text{tension}} \) for the cases covered by the sensitivity analysis is a reasonable one. When a thicker tube is used for the column,
150×9SHS, the tension region is approximately four to seven times stiffer than the compression region. Although this is the case, adopting the assumption of $K_{\text{compression}} = 5K_{\text{tension}}$ is reasonable as $K_{\text{tension}}$ is underpredicted by the component model and the overall prediction of the connection stiffness is conservative as shown in Table 7-4 (see discussion below).

Table 7-3: Comparison of connection stiffness in compression region to tension region (FE model)

<table>
<thead>
<tr>
<th>Model</th>
<th>Endplate thk (mm)</th>
<th>Backplate thk (mm)</th>
<th>Stem thk (mm)</th>
<th>Channel thk (mm)</th>
<th>$K_{\text{tension}}$</th>
<th>$K_{\text{compression}}$</th>
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<tr>
<td>150×9 SHS</td>
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Table 7-4 compares the prediction from the component model to the FE prediction for the overall connection stiffness based on Equation 7-7. The connection is rigid for braced frames when a thick endplate and backplate (20mm) and a thick channel (9mm) are used. For rigid conditions, the maximum error between the FE prediction and component model is 15%. When a thinner endplate and backplate (10mm) and / or a thinner channel (6mm) are employed, the connection becomes semi-rigid. The component model underpredicts the FE model with a higher percentage error (20-36%) when the connection is semi-rigid.

This configuration is not recommended as a semi-rigid connection due to the complexity of the configuration and various numbers of components required. For a semi-rigid connection, other less complex configurations can be adopted.
Hence the higher percentage error in the component model when predicting the
connection stiffness for the semi-rigid condition becomes less relevant. The
applicability of this model to connections with dimensions different to those
studied here requires further investigation.

<table>
<thead>
<tr>
<th>Model</th>
<th>Endplate thk (mm)</th>
<th>Backplate thk (mm)</th>
<th>Stem thk (mm)</th>
<th>Channel thk (mm)</th>
<th>$K_{FE}$ (kN/mm)</th>
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<th>Backplate thk (mm)</th>
<th>Stem thk (mm)</th>
<th>Channel thk (mm)</th>
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<td>7828</td>
<td>0.73</td>
<td>Semi-Rigid</td>
</tr>
</tbody>
</table>

### 7.6 Summary and conclusions

A new blind bolted connection to unfilled SHS column was proposed and
investigated. This new connection is a modification of a typical T-stub connection,
with additional channels connected to an extended endplate. The channels help to
transfer loads to the back face of the SHS column in bearing; thereby reducing
demand on the flexible column face. This results in a much stiffer connection in
comparison to the typical T-stub connection. The findings from the test and
subsequent analysis may be summarised as below:

1) The experimental results show that the extended T-stub blind bolted
connection with back face support has the potential to be classified as a
rigid connection for a braced frame system according to the Eurocode 3
specifications. The stiffness of the connection is not compromised by the
flexible hollow section column face deformation normally observed in typical face connections.

2) The extended T-stub connection with back face support was found to be comparable to the blind bolted channel side plate connection and is much stiffer than a typical face connection. Comparing the stiffness of the extended T-stub connection with back face support tested in this chapter with a 10mm thick T-stub connection for the same size tube and beam, the extended T-stub connection with back face support is approximately five times stiffer than the T-stub connection and is 2.5 times stiffer when the same endplate thickness is used. This highlights the benefit of using the channel sections to transfer load to the back face of the SHS column as compared to the conventional front face connections.

3) An extensive 3-D non linear finite element modelling was developed for the extended T-stub connection with back face support. The model consisted of various components such as the extended T-stub, beam, channels, tube and backplate and also various contact surfaces between these components. This extensive 3-D FE model was able to replicate the behaviour of the test specimen successfully.

4) Based on the sensitivity analyses performed using finite element modelling, the T-stub and backplate thickness were found to be crucial for the connection stiffness. A thinner plate is more flexible in bending hence compromises the overall connection stiffness. Channel thickness also affects the connection stiffness, the thinner the channel the easier it is for the corners to open up when subjected to loading. A thicker tube has higher face stiffness when subjected to tensile loading from bolts hence resulting in an overall stiffer connection.
5) A simplified component model was developed in this chapter to predict the initial stiffness of the connection which governs the classification of the connection. While the component model results are conservative, it is quite simple to follow. When the connection is rigid for braced frames, the component model underpredicts the FE stiffness within 15% accuracy. However, the component model underpredicts the FE stiffness by a maximum error of 36% when the connection is semi-rigid. The extended T-stub connection with back support is not recommended as a semi-rigid connection due to the various components required. Other simpler configurations could be used for a semi-rigid connection.

6) The extended T-stub and back face support does not require blind bolts to be pretensioned. This is advantageous in terms of simplifying installation procedures on site as compared to the channel side plate connection discussed in Chapter 6 which requires all blind bolts to be pretensioned. With the various types of blind bolted connections investigated in this thesis, a designer can mix and match these connections to suit a design situation. For example, when there is no reversal of moment, the extended T-stub with back face support or the channel side plate connection can be used at the top while the typical T-stub connection can be used as the bottom connection. This is discussed further in the following chapter.
Chapter 8

Discussions and Applications

8.1 Introduction

This chapter collates the findings from the preceding chapters of this thesis; provides suggestions for connection configurations in the secondary framing direction and practical construction procedures, and provides a case study to investigate the influence of connection stiffness on an overall frame stiffness.

8.2 T-stub connection

The T-stub connection is a typical face connection which provides a simple and direct method of connecting open section beams to hollow section columns. Very few components are required; the T-stubs can be obtained from commercially available universal or welded beams (UB or WB) sections or by welding two plates together to form the T-stub. The former method requires less fabrication work and is the preferred option for mass production.
The order of construction for the T-stub connection is straightforward, initially placing the bottom T-stub in place by blind bolting to the hollow section column. Next, the beam is lifted into position and is seated on the bottom T-stub. The bottom flange of the beam is bolted to the T-stem. The top T-stub is then blind bolted into position and the top flange of the beam bolted to the top T-stem. Illustration of the construction sequence is shown in Figure 8-1.

**Figure 8-1: Construction sequence for T-stub connection**

The T-stub connection can be used on all four sides of a column as shown in Figure 8-2. This provides connections in all directions of the columns. Alternatively, a simple angle cleat connection could be used for beams in the secondary direction if they are designed as pin ended as illustrated in Figure 8-3. The angle cleats are blind bolted to the tube face and connected to the beam webs in the secondary direction. Diagonal bracing members or shear walls are required in the secondary direction for lateral stability when pinned connections such as angle cleats, welded fin plate or flexible endplate are used. Design procedures for these pinned connections are well documented in the BCSA/SCI: Joints in Simple Construction (2002) and AISC: Design of Structural Connections.
Blind Bolted Connections for Steel Hollow Section Columns

(Hogan & Thomas, 1994). An example to design the simple angle cleat and flush endplate connection with Ajax ONESIDE blind bolt is given in Appendix C.

Despite its versatility and ease of construction, the T-stub connection was found to be well and truly semi rigid from the findings in Chapter 4. Increasing the endplate thickness helps to improve the stiffness of the connection up to a certain extent, beyond which the tube face’s inherent flexibility governs. Doubling endplate thickness from 10mm to 20mm for a 150×6SHS and a 310UB32 beam results in an improved stiffness from 3000kNm/rad to 5000kNm/rad. The 20mm thick endplate forms the upper boundary of the connection stiffness which is well below the designated rigid boundary for braced frames of 16850kNm/rad (for 6m span 310UB32 beam) according to the Eurocode 3 classifications (2005).

Figure 8-2: T-stubs connection on all four directions
8.3 Side connections to hollow section column

In order to address the issue of a flexible column face, the concept of connecting to the side walls of the hollow section column was introduced. Chapter 5 explored this initial concept by implementing a collar connection to the SHS column in the tension region. The 20mm thick collar plate connection showed very promising results in which the flexible column face deformation was fully avoided. The collar plate requires extensive welding during fabrication which is not an attractive option where fabrication costs are high. Further, different collars have to be made for different column sizes. Further development built on the results from the collar plate connection leading to the introduction of the channel side plate connection in Chapter 6 which utilises readily available commercial sections. The channel components can be cut from the SHS column itself or from a thicker tube of the same size, while the side plates are cut from standard flats.

Compared to the T-stub connection, the channel side plate connection has the same number of blind bolts but requires an additional number of standard bolts to connect to the channel legs. Stiffness of the channel side plate connection for a
150\times 6\text{SHS} column and 310\text{UB32} beam, 6\text{mm} thick channel and 12\text{mm} thick side plate as tested in Chapter 6 was 17000\text{kNm}/\text{rad} which could be increased by 25-30\% if a 9\text{mm} channel was used instead of 6\text{mm} or if the coefficient of friction was increased from 0.15 (painted surfaces) to 0.25 (close to black steel).

All bolts in the connection assembly must be pretensioned to obtain the desired stiffness. Pretensioning of the blind bolts in practice can be achieved using the part turn method as proposed in the Australian Steel Code, AS 4100 (Standards Australia, 1998) although further calibration of the ONESIDE blind bolt with this method is recommended. Alternatively a load indicating washer could be used whereby the dimples on the washer will indent the mating surface when tension is applied to the bolt. These washers have to be used with standard washers supplied with the Ajax blind bolt. The Ajax washer is necessary as it is a stepped washer and in part fills the gap between the bolt hole and bolt shank. Another type of load indicating washer is the Squirter® washer by Hobson Engineering which operates as a typical load indicating washer and has the added advantage of allowing visual inspection of the bolt assembly when it is tensioned correctly by “squirting” bright orange silicone.

The construction sequence for the connection has been discussed in Chapter 6 whereby channel sections are first bolted to the beam section. The side plates are then bolted to the channels on one side, and on the tube side wall on the other side, before bringing the tube and the beam together. The remaining bolts on the side plates are then locked in place as illustrated earlier in Figure 6-1. However, if there is no reversal of moment in the connection, a T-stub could be used to replace the channel and the side plates at the bottom of the beam. This provides a simpler construction sequence as illustrated in Figure 8-4. The channel is first bolted to the top flange of the beam, the T-stub is then connected to the column face at the bottom, beam is placed into position and finally side plates are introduced to tie the channel and tube side wall together.
The FE model in Chapter 6 was modified to substitute the compression region channel side plate connection at the bottom of the I-beam with a 10mm thick T-stub as shown in Figure 8-5. A comparison of moment rotation curves between the default connection and the modified connection with T-stub at the bottom of the beam is plotted in Figure 8-6. Both connections have a very similar behaviour which indicates that the T-stub could easily replace the channel side plate connection in the compression region when reversal of moment is not expected in the design of the overall connection.

Figure 8-4: Construction sequence for a combined site plate and T-stub connections
Figure 8-5: FE model of channel side plate and T-stub connection combination

Figure 8-6: Moment rotation curve for combination of channel side plate and T-stub connection
If channel side plate connections are used in the primary direction, the connection in the secondary direction can be a pinned or a moment connection. A pinned connection in the secondary direction would be similar to the one proposed for the T-stub connection i.e. angle cleats as shown in Figure 8-7, or a welded fin plate or flush endplate connection. If moment resisting connections are required in the secondary direction, the inverted angles and plate connection can be adopted as shown in Figure 8-8. Angle sections are bolted to the inside of the beam flanges and a long plate is then inserted through the gap between the primary beam and the tube face to connect to the inverted angles. These schematic connections for the secondary direction have been developed to examine the geometric and construction feasibility. No analysis or testing has been performed on these connections.
8.4 The extended T-stub connection with back face support

The extended T-stub connection with back face support was proposed with the aim to achieve comparable stiffness as the channel side plate connection without having to pretension the blind bolts. The extended T-stub connection with back face support has additional components compared to a typical T-stub connection.
Channels are bolted to the extended T-stub to distribute beam flange tension load from the tube front face to the backplate which bears on the back face of the tube. This reduces the load demand on the flexible tube face. It is desirable to use a thicker channel than the column thickness to maximise the load sharing between the channels and the tube face. The extended T-stub connection with back face support tested in Chapter 7 with a 20mm thick endplate has an initial stiffness 17000kNm/rad which is comparable to that of the channel side plate and much greater then the 5000kNm/rad achieved by the 20mm thick typical T-stub connection.

The construction sequence for the extended T-stub connection with back face support is similar to that for a typical T-stub connection. The bottom T-stub is first connected to the tube face, and then the beam is seated on the bottom T-stub. The top T-stub is bolted into position before the channels and backplates are connected together as illustrated in Figure 8-9.

![Figure 8-9: Construction sequence for extended T-stub connection](image)

Connection in the secondary direction to the extended T-stub with back face supports connection has its complications due to the protrusions from the channel
legs. A moment connection in the secondary direction can be achieved with a T-stub connection. The width of the T-stub has to be confined within the clear space inside the channel. If the secondary beam flange is wider than the clear space within the channel, the flange has to be coped as illustrated in Figure 8-10.

![Figure 8-10: Moment connection in secondary direction](image)

On the other hand, a pinned connection can be achieved by a welded fin plate as illustrated in Figure 8-11. The fin plates would have to be prewelded on the channels off site. Flanges of the secondary beams may need to be coped to fit between the channel legs. Further study is needed to consider the interaction between the primary and secondary directions.
8.5 Case Study: Influence of connection stiffness on overall frame stiffness

A three-storey steel framed building on a soft soil site, soil class E, AS 1170.4 (Standards Australia, 2007) was considered as a case study to investigate the effect of connection stiffness in the overall frame stiffness. The frame dimensions are given in Figure 8-12. The single frame was modelled using the commercially available frame analysis software, SPAGE GASS (2011).

The frame is considered to be a lightweight frame subjected to a superimposed dead load (SDL) of 1.5kPa and live load (LL) of 3kPa for levels 1 and 2, while the roof is subjected to SDL of 0.5kPa and LL of 0.25kPa based on the Australasian loading code AS/NZS 1170.1 (Standards Australia, 2002b). The assumed spacing between frames is 4m.

The following load combinations were used for the frame design based on the Australasian loading code, AS/NZS 1170.0 (Standards Australia, 2002a):
(i) 1.2G + 1.5Q
(ii) 1.2G + W_u + 0.4Q
(iii) 0.9G + W_u
(iv) G + E_u + 0.3Q
(v) G - E_u

where G = dead load, Q = live load, W_u = ultimate wind load and E_u = earthquake load

The connections between the columns (150×6SHS) and beams (310UB32) were modelled as springs as shown in Figure 8-12. The stiffness of the springs was varied from that representing a rigid connection to that representing a pinned connection.

The frame was found to be satisfactory in terms of strength and deflection check under gravity loading. The connections are required to transfer a maximum ultimate moment of 35kNm (for fully fixed condition) and a serviceability moment of 20kNm. The connection can be designed to withstand serviceability (or ultimate) moment without slip.

Under lateral loading, the frame was checked for both earthquake and wind loads. The frame was designed for earthquake design category importance level 2 for an ordinary moment resisting frame based on the Australian earthquake code, AS 1170.4 (Standards Australia, 2007). The frame was assumed to be an ordinary moment resisting frame with ductility, µ of 2. The frame was designed according to Clause 5.4.2.3 from AS 1170.4 for structures not exceeding 15m tall.

Force per floor, F_i = K_s[k_p Z_{S_p}/\mu]W_i

where
k_p = probability factor (1.0 for 1/500 annual probability of exceedance)
Design base shear for earthquake was 26kN. The earthquake load was compared to a wind load of 1kPa based on the Australasian wind code, AS/NZS 1170.2 (Standards Australia, 2002c). Wind load was found to be the governing lateral load.

The interstorey drift of the frame was compared for different spring stiffnesses under imposed wind and earthquake loads. The interstorey deflections for serviceability wind conditions were checked against the suggested drift limit of $h/500$ in AS/NZS 1170.0 (Standards Australia, 2002a) where $h$ is the storey height. Under earthquake load, the storey drift from elastic analysis was multiplied by $\mu/S_p$ of 2.6 for ordinary moment resisting frames where $\mu$ is the structural ductility factor and $S_p$ is the structural performance factor; the storey drift was checked against the allowable drift limit of 1.5% storey height from the earthquake code, AS 1170.4 (Standards Australia, 2007). Interstorey deflections due to serviceability wind were found to be more critical than earthquake hence the comparison results for wind loading are tabulated in Table 8-1.
In this particular case study, bracing was required on level 1 to satisfy the drift limit of $h/500$ even when the connections were infinitely stiff (rigid). For level 2, connection stiffness of 10000kNm/rad and above was required to satisfy the drift limit. This minimum stiffness requirement for level 2 was easily met by the stiffness of the proposed connections in Chapter 6 (channel side plate connection) and Chapter 7 (extended T-stub with back face support connection): both having connection stiffness of 17000kNm/rad from experimental findings. The simple T-stub connection in Chapter 4 with stiffness of 3000 to 5000kNm/rad depending on T-stub thickness could not satisfy the stiffness demand for level 2. The connection stiffness required to satisfy level 3 interstorey deflections was less demanding. No bracing was required for levels 2 and 3 in the case study with a minimum stiffness of 5000kNm/rad.
Table 8-1: Interstorey drift of frame for different connection stiffness

<table>
<thead>
<tr>
<th>Level</th>
<th>Storey Height (mm)</th>
<th>Interstorey drift (mm)</th>
<th>K_{connection} (kNm/rad)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>K=rigid</td>
</tr>
<tr>
<td></td>
<td></td>
<td>δ</td>
<td>δ/(h &lt; 1/500)</td>
</tr>
<tr>
<td>3</td>
<td>3000</td>
<td>1.9 OK</td>
<td>2.2 OK</td>
</tr>
<tr>
<td>2</td>
<td>3000</td>
<td>4.6 OK</td>
<td>5.1 OK</td>
</tr>
<tr>
<td>1</td>
<td>3500</td>
<td>4.4 OK</td>
<td>4.5 OK</td>
</tr>
</tbody>
</table>

8.6 Summary

1) In this chapter practical construction sequences have been proposed for the new connections discussed in the preceding chapters.

2) Proposals have also been made for connections in the orthogonal secondary direction to the T-stub, channel side plate and the extended T-stub with back face support. It should be noted that no tests have been conducted for these connections. Further study is required to consider the interaction between the connections in the primary and secondary directions.

3) When there is no reversal of moment in the connection, a simple T-stub connection could be used at the bottom of the beam for an overall simpler construction sequence and installation.

4) A sample three-story steel framed building has been analysed for various load combinations for different connection stiffnesses (from pinned to rigid). Enhanced connection stiffness is shown to have a significant influence on the lateral serviceability performance of the frame and assist in
eliminating the need for bracing members in the frame or in reducing the size of bracing members required.
Chapter 9

Conclusions and Recommendations

9.1 Summary and contributions

A research program has been conducted at the University of Melbourne to investigate different configurations of blind bolted connections to hollow section columns utilising the Ajax ONESIDE blind bolts.

The successful development of the proposed blind bolted connections offers a convenient and efficient alternative for beam-to-hollow section column connections. For the first time in Australia, a fully bolted moment-resisting site connection between open section beams and hollow section columns is achievable, providing a reliable and efficient construction solution. This has the potential to facilitate the development of economical and innovative framing options using hollow sections in the low rise construction market.
The rigidity developed by the connections will enable longer floor spans i.e. increasing column free spaces in both residential and commercial buildings without compromising floor stiffness and serviceability performance related to deflections and floor vibrations. Furthermore, the ability to quantify the stiffness of the connections makes it possible for the connections to be designed as semi-rigid. The development of simplified design models in this thesis for the connections will facilitate the adoption of the proposed blind bolted connections by practising engineers.

9.2 General conclusions

A summary of key findings from the research is as follows:

1) A new blind bolt, Ajax ONESIDE, can be used to develop bolted connection alternatives to welded connections in residential and low rise commercial frames utilising hollow section columns.

2) Bolt sleeves are recommended to be used as part of the ONESIDE assembly. They help to improve shear capacity of the bolt and shear stiffness of the connection. The sleeves also provide compensation for the oversize bolt hole. Their efficiency is dependent on the tolerance between the outer diameter of the sleeves and bolt hole. The tighter the tolerance, the more efficient the sleeves are in reducing bending of the bolts. They also help reduce slip between the endplate and the tube face in the case of face connections, and slip between the side plate and the tube in the case of side connections.
3) Pretension of the bolts in the connection helps to improve stiffness of the clamped surfaces. The effective clamped area is generally limited to the washer area.

4) The T-stub connection is a typical face connection, connecting to the face of the tube. It has minimal components and is simple to install. Based on testing and FE analyses, it has been found that this type of connection is semi-rigid due to the inherent flexibility of the tube face. Increasing the endplate thickness helps to improve the connection stiffness up to a certain point beyond which the flexible deformation of the tube face governs. A thicker tube leads to a dramatic improvement in the connection stiffness. For practical commercially available sizes of tubes in Australia, the tube wall thicknesses would not be large enough to achieve rigid status.

5) As connection to the tube face is limited by the flexible column face deformation, a pilot test using a collar plate connection which connects to the side walls of the tube was carried out and presented in this thesis. The collar plate connection exhibited a much improved strength and stiffness compared to the T-stub connection giving confidence that connecting to the sides of the tube is a feasible option for achieving greater rigidity. While the collar connection can achieve typical design requirement for strength and stiffness, it requires significant fabrication.

6) A newly developed channel side plate connection, which is a simplification of the collar plate in terms of fabrication, has been presented. The merit of this connection is that it does not require any welding; all parts of the connection can be sourced from commercially available sections. This connection is dependent on the depth of the channel being equal to the width of the SHS column (channel obtained
by cutting the SHS column section into halves), in order that the side plates can bolt into both channel flange and SHS wall. Based on full scale testing and extensive FE analyses, the channel side plate connection can be designed as a rigid connection for braced frames according to the Eurocode 3 classifications. The connection stiffness can be further enhanced through a number of options. These include: using a bigger bolt size (hence a higher pretension load in the bolts), increasing the coefficient of friction between connecting surfaces by removing the surface paint with hand wire brushing or light abrasive brushing, and by using a thicker size channel. The effective pretensioning of all bolts is critical for this connection. The high stiffness of the connection is largely dependent on the clamping action of all the bolts. Once slip commences, the connection would have a much reduced stiffness. However, the ultimate strength of the connection is maintained.

7) An alternative connection which has no reliance on pretension in the blind bolts has also been developed. The extended T-stub with back face support is a modification of the typical T-stub connection. The extended T-stub is connected to channels which are tied back to the back face of the tube with a backplate. The channels help to distribute the tension load on the tube face (from the beam flange) and transfer this tension load to the back face of the tube via bearing action. The blind bolts are not required to be pretensioned in this connection as they are acting mainly in tension. All other standard bolts are required to be pretensioned. A 20mm thick endplate could match the stiffness of the tested channel side plate connection and can be considered as rigid according to the Eurocode 3 specifications. The stiffness of the connection is largely dependent on the channel, endplate and backplate thicknesses. The ultimate strength of the connection is substantially higher than the T-stub or the channel side plate connection.
8) A designer could mix and match the proposed blind bolted connections for the top and bottom beam flanges to suit different design situations. For example when no reversal of moment is present, the channel side plate or extended T-stub connection with back face support can be used as the top connection to maximise stiffness while the typical T-stub connection can be used as the bottom connection for simple construction installation.

9) Detailed three-dimensional non-linear FE models have been developed for each of the proposed connections, simulating boundary conditions and applied load during experimental testing. Advanced contact interactions between various surfaces have been modelled and initial pretension loads in the bolts have been incorporated using sophisticated pretensioned sections. The FE models have been found to give very good agreement to the experimental results and have been used for extensive parametric studies.

10) Simplified design methodologies for the typical T-stub connection, channel side plate connection and extended T-stub connection with back face support have been developed to predict the strength and stiffness of such connections. Strength design for these connections can be found largely in available design standards and guidelines. The stiffness of each connection is determined using the component method proposed by Eurocode 3 whereby springs are used to represent the stiffness of individual components in the connections. The assembly of the springs’ stiffnesses gives the overall connection stiffness. The simplified theoretical models give good approximation to the FE and experimental results and provide a fast and convenient way for designers to calculate the stiffness of the proposed connections.
11) Practical construction sequences have been proposed for the connections discussed in this thesis. Proposals have also been made for connections in the orthogonal secondary direction to the T-stub, channel side plate and the extended T-stub with back face support.

12) Finally, a typical three-storey steel framed building has been chosen as a case study; investigating the effects of varying connection stiffness (from pinned to rigid) on the lateral stability and serviceability performance of the frame. Connection stiffness is shown to provide great benefits in assisting the overall stability of the frame and reducing or eliminating the need for bracing members.

9.3 Recommendations for further study

The following items are recommendations for future work which would extend the applicability of the research findings:

1) The columns size tested in the experimental programme is the 150×150×6 SHS which is a commonly used member size for low rise commercial applications. Detailed 3-D FE models have been generated to simulate the behaviour of the proposed connections under static loading and have been found to give accurate and reliable predictions. The connections proposed can be used for other column sizes as well for heavier or lighter industry. It is recommended to conduct additional experimental tests to further validate the FE and theoretical predictions for the 89×89SHS commonly used in residential construction and for column sizes bigger than 200×200SHS. This is also important for cases where the ratio of
beam to column stiffness, $I_b/I_c$ exceeds 6 (the stiffness ratio of the tested 310UB32 beam to 150×150×6 SHS is approximately 6 times).

2) The 89×89SHS columns are commonly used in residential dwellings to fit into the stud walls of a typical house frame in Australia. The current practice to achieve a moment connection with these columns is via welding of angle sections to the SHS for beams to be connected as discussed in Chapter 2. In order to extend the use of the proposed blind bolted connections to these small size columns, the geometrical limitations required for the installation of ONESIDE blind bolt needs to be investigated. A modification to the current installation tool is needed to reduce the clear space required by the tool inside the hollow section columns for installation of the blind bolts. AJAX have developed a number of concepts to achieve this, however, prototype tools are yet to be developed.

3) The potential influence of axial compression load in the SHS column on the capacity of the blind bolted connections should be investigated. Numerical studies could be undertaken initially followed by experimental testing for cases where the numerical results indicate that it could be most significant.

4) The LiteSteel® beam (LSB) has become increasingly popular in Australia for light weight construction of residential, warehouses, and mezzanine levels. These channel sections have hollow flanges. It is proposed to investigate the connectivity of these sections to the hollow section columns with some modifications to the proposed blind bolted connections in this thesis.
5) It is also recommended to develop span tables for typical framing grids to cover deflections, vibrations and lateral sway.

6) Cyclic testing is recommended for the proposed blind bolted moment connections to explore any degradation in strength and stiffness under cyclic deformations for required conditions in the Australian design.
References


Blind Bolted Connections for Steel Hollow Section Columns


Appendix A

Proposed strength design of T-stub connections for SHS columns

A quantitative procedure for designing T-stub connections for hollow section columns using Ajax ONESIDE blind bolts is proposed in this appendix. The design criteria for the blind bolted T-stub connection based on the recommendations by Mourad (1994) are:

1) The connection deformation should satisfy the allowable structural drift limits.

2) Failure of blind bolts and welds (if any) before either the T-stub endplate, column face or the beam reach its plastic capacity and experience strain hardening, should be prevented.

3) The column face thickness should be adequate to prevent punching failure of the Ajax ONESIDE blind bolts under tension loading.
4) No significant inelastic action should take place in the T-stub endplate to avoid brittle failure in the heat affected zone if the T-stubs are fabricated from welded plates.

5) As the in plane stiffness of the SHS column side walls is greater than the out of plane stiffness of the column face, it is recommended to place the blind bolts as close as possible to the side walls to transfer forces more directly from the beam flange to the side walls to prevent excessive column face deformation.

6) In order to minimise endplate deformation and prying forces, the blind bolts should be located as close as possible to the T-stem (distance ‘m’ in Figure 4-34a) while satisfying installation requirements to accommodate impact wrenches (Hogan & Thomas, 1994). Mann and Morris (1979) also suggested that the distance between the exterior blind bolts and the edge of the endplate (distance ‘n’ in Figure 4-34a) should be larger or at least the same as distance ‘m’ to minimise prying action.

A.1 Design Steps

In this section, the design procedure of a T-stub connection for 310UB32 beam and 150×150×6 SHS column is presented. Figure A-1 denotes the various notations used in this appendix.

Assumptions:
- Frame grid of 6m × 4m
- 310UB32 beam with section capacity, \( \phi M_p,_{beam} = 134\text{kNm} \)
- 150×150×6 SHS with column section capacity, \( \phi M_p,_{column} = 55\text{kNm} \)
M16 (16mm diameter) 8.8/TF standard bolts for T-stem to beam flange connection, bolt hole 18mm diameter

M16 8.8/S blind bolts for endplate to column face connection, bolt hole 24mm diameter

Material properties

- 150×150×6 SHS column
  \[ f_y = 350 \text{MPa} \]
  \[ f_u = 430 \text{MPa} \]
  where \( f_y \) = yield strength and \( f_u \) = ultimate strength of member

- 310UB32 beam and T-stub
  \[ f_y = 300 \text{MPa} \]
  \[ f_u = 440 \text{MPa} \]

Design Actions

Superimposed dead load + self weight = 1.5kPa

Live load = 4kPa

Design shear force, \( V^* \)

\[ V^* = \frac{[(1.2 \times 1.5 + 1.5 \times 4) \times 4] \times 6}{2} = 94kN \]

Design connection to have moment capacity that matches the lower of the beam or column capacity, hence \( M^* = 55 \text{kNm} \) (as per column’s capacity)

Step 1: Determine design force in T-stem, \( F^* \)

\[ F^* = \frac{M^*}{d_{blm}} \]

\[ F^* = \frac{55}{0.32} = 172kN \]
**Step 2: Check for column face yielding and pull-out action of blind bolts from tube face, revise \( F \) in Step 1.**

Thin column face will normally govern the design for T-stub connection to hollow section columns as discussed earlier in Chapter 4.

From Equation 4-1 in Chapter 4 (Kurobane et al., 2005), column face plastification, \( N_{pl} \) (using oversized bolt hole diameter, \( d_h \)) is given by:
Blind Bolted Connections for Steel Hollow Section Columns

\[
N_{pl} = \frac{f_{c,y} t_c^2 \left[ \frac{2(h_b - d_h)}{b'} + 4 \sqrt{1 - \left( \frac{c}{b'} \right)} \right]}{1 - \left( \frac{c}{b'} \right)}
\]

(A-2)

Hence,

\[
N_{pl} = \frac{350 \times 6^2 \left[ \frac{2(150 - 24)}{(150 - 6)} + 4 \sqrt{1 - \left( \frac{70 - 24}{(150 - 6)} \right)} \right]}{1 - \left( \frac{70 - 24}{(150 - 6)} \right)} \times 10^{-3} = 93kN
\]

Note that \( N_{pl} < F^* \), hence \( F^* \) can be revised to match \( N_{pl} \), let design force in T-stem, \( F^* = 1.1N_{pl} \approx 110kN \)

The blind bolts will also need to be checked for pull-out from column tube face. Equation A-3 is given in CIDECT Design Guide 9 for Hollobolt punching shear capacity (Kurobane et al., 2005).

\[
F_{ps} = 0.6f_{c,y} \pi t_c d_b
\]

(A-3)

where

- \( f_{c,y} \) = column yield stress
- \( t_c \) = column thickness
- \( d_b \) = diameter of bolt

Hence

\[
F_{ps} = 0.6 \times 350 \times \pi \times 6 \times 16 \times 10^3 = 63kN \text{ per bolt}
\]

Total \( F_{ps} = 4 \times 63 = 252kN > N_{pl} \), hence column face plastification will occur before punching shear failure of bolt as expected.

The blind bolts also need to be checked against bearing action (as per Equation A-10), although this is not critical.
**Step 3: Check thickness of stem, \( t_{stem} \), required to transmit tension force in the T-stem**

Plate in tension

\[
\Phi N_t = \text{lesser of } 0.9A_gf_y \quad \text{Clause 7.1 – AS4100 (A-4)}
\]

and \( 0.9(0.85)k_tA_nf_u \)

where

- \( A_g \) = gross area of cross section
- \( A_n \) = net area of cross section
- \( k_t \) = a factor for eccentricity of loading, 1.00 for uniform force distribution

\[
0.9A_gf_y = 0.9 \times 150 \times 20 \times 300 \times 10^{-3} = 810kN
\]

\[
0.9(0.85)k_tA_nf_u = 0.9 \times 0.85 \times (150 \times 20 - 20 \times 18 \times 2) \times 440 \times 10^{-3} = 767kN
\]

Hence \( \Phi N_t = 767kN > F^* = 110kN \)

The T-stem is also subjected to bending action, \( \frac{V^*}{2} \times e \) as shown in Figure A-2 where \( \frac{V^*}{2} \times e = \frac{94}{2} \times 0.05 = 2.4kNm \).

Plate in bending

\[
\Phi M_s = 0.9f_yZ_e \quad \text{(A-5)}
\]

where

- \( Z_e \) = effective section modulus

\[
\Phi M_s = 0.9 \times 300 \times (150 \times 20^2/6) \times 10^{-6} = 2.7kNm > \frac{V^*}{2} \times e \quad \text{hence T-stem is satisfactory in bending.} 
\]
Step 4: Determine minimum number of blind bolts, \( N_{bb} \) required to carry tension force to column face, check blind bolts for shear and combined actions.

Hogan (1994) suggests allowing for 30% prying action in the bolts.

Assume M16 bolts are used

Blind bolt tension capacity,

\[
\phi N_{tf} = 0.8A_s f_{uf} \quad \text{Clause 9.3.2.2 – AS4100 (A-6)}
\]

where

\( A_s \) = tensile stress area
\( f_{uf} \) = minimum tensile strength of the bolt

\[
\phi N_{tf} = 0.8 \times 157 \times 830 \times 10^{-3} = 104 \text{kN per M16 bolt}
\]

Hence \( N_{bb} = \frac{1.3 F^*}{\phi N_{tf}} = \frac{1.3 \times 110}{104} = 1.4 \), therefore adopt \( N_{bb} = 4 \) bolts

Blind bolt shear capacity,

\[
\phi V_f = 0.8(0.62)k_i f_{uf}(n_a A_c + n_v A_o) \quad \text{Clause 9.3.2.1 – AS4100 (A-7)}
\]
where

\( n_n \) = No. of shear planes with threads included
\( n_x \) = No. of shear planes with threads excluded
\( A_c \) = bolt minor area
\( A_o \) = nominal shank area
\( k_r \) = lap length factor, 1.0 for lap length \( \leq 300\text{mm} \)

Assume threads are included; sleeve of blind bolt will contribute to \( A_c \)
Outer diameter of sleeve = 22.5mm, inner diameter = 16.5mm

\[ \Phi V_{fb} = 0.8 \times 0.62 \times 1.0 \times 830 \times (1 \times 327) \times 10^{-3} \]
\[ = 134\text{kN per M16 blind bolt} \]

There are a total of 8 blind bolts (top and bottom T-stubs) to carry shear
\( V_f^* \) per blind bolt = 94/8 = 12kN, each blind bolt has ample capacity in shear

Check for combined actions

\[ \left( \frac{V_f^*}{0.8V_f} \right)^2 + \left( \frac{N_{f_s}^*}{0.8N_{f_s}} \right)^2 \leq 1.0 \quad (A-8) \]

\[ \left( \frac{12}{134} \right)^2 + \left( \frac{36}{104} \right)^2 = 0.13 < 1.0 , \text{ therefore the blind bolt satisfies combined action} \]

**Step 5: Determine minimum number of standard bolts, \( N_{sb} \), to transfer shear force in the T-stem**

Use Equation A-7 to determine shear capacity of a standard 8.8 M16 bolt

\[ \Phi V_{fsb} = 0.8 \times 0.62 \times 1.0 \times 830 \times (1 \times 144) \times 10^{-3} \]
\[ = 59\text{kN per M16 standard bolt} \]

\[ N_{sb} = \frac{F^*}{\phi V_{f-sb}} = \frac{110}{59} = 1.9 , \text{ therefore adopt } N_{sb} = 4 \text{ bolts} \]
Number of bolts on the T-stem is also dependent on the serviceability load that the connection is expected to withstand without slip.

Assume serviceability load, \( F_{\text{serv}} = 0.6 \times F^* = 66 \text{kN} \)

Bolts in friction grip (TF)

\[
\Phi V_{sf} = 0.7 \mu_k h_n N_{ti} \quad \text{Clause 9.3.3 – AS4100} \quad (A-9)
\]

where

- \( \mu \) = slip factor (assume to be 0.15 as per slip test results from Section 6.5.2.1)
- \( k_h \) = hole factor
- \( n_{ei} \) = No. of effective interfaces
- \( N_{ti} \) = minimum bolt tension at installation, 95kN for M16 bolt

\[
\Phi V_{sf} = 0.7 \times 0.15 \times 1 \times 95 = 10 \text{kN}
\]

\[
N_{sb} = \frac{0.6 F^*}{\Phi V_{sf}} = \frac{66}{10} = 6.6, \text{ hence adopt 8 bolts to satisfy serviceability load condition.}
\]

Bolts on the T-stem will also need to be checked for ply bearing and tearing, although this is usually not critical.

Bolts in bearing

\[
\Phi V_b = \text{lesser of } 0.9 a_e t_p f_{up} \quad \text{Clause 9.3.2.4 – AS4100} \quad (A-10)
\]

and \( 0.9 (3.2) d_b t_p f_{up} \)

where

- \( a_e \) = edge distance
- \( t_p \) = ply thickness
- \( d_b \) = diameter of the bolt

\[0.9 a_e t_p f_{up} = 0.9 \times 30 \times 20 \times 440 \times 10^{-3} = 238 \text{kN}\]

\[0.9 (3.2) d_b t_p f_{up} = 0.9 \times 3.2 \times 16 \times 20 \times 440 \times 10^{-3} = 405 \text{kN}\]

Hence \( \Phi V_b = 238 \text{kN} \) per bolt which satisfies the shear force on the T-stem easily.
Step 6: Determine T-stub endplate thickness, taking prying action into account.

Hogan (1994) suggests the following equations to design the endplate for flexure and shear. Flexure design is based on flexible T-stub flange assumption as shown in Figure A-3 where plastic hinges are developed at the blind bolt lines and at the T-stem-to-endplate intersection.

Flexure capacity of endplate

\[ \Phi N_{ep} = 0.9 f_{y, ep} b_{ep} t_{ep}^2 / a_{ie} \]  \hspace{1cm} (A-11)

where

- \( a_{ie} \) = \( m \) in Figure A-3 for butt welds
- \( f_{y, ep} \) = yield stress of endplate

Let \( \Phi N_{ep} = F^* \)

\[ t_{ep} \geq \sqrt{\frac{F^* m}{0.9 f_{y, ep} b_{ep}}} \], therefore \( t_{ep} \geq \sqrt{\frac{110 \times 1000 \times 65}{0.9 \times 300 \times 150}} = 13.3 \text{mm} \)

Hence adopt \( t_{ep} = 16 \text{mm} \) to satisfy \( F^* = 110 \text{kN} \)

Check shear capacity of endplate for horizontal and vertical shear.

\[ \Phi V_{ep_v} > V^* \]  \hspace{1cm} (A-12)
\[ \Phi V_{ep_h} > 2F^* \]

where

- \( \Phi V_{ep_v} = 0.9 \times (0.5 f_{y, ep} l_{ep} t_{ep}) \)
- \( \Phi V_{ep_h} = 0.9 \times (0.5 f_{y, ep} b_{ep} t_{ep}) \)

Hence

- \( \Phi V_{ep_v} = 0.9 \times (0.5 \times 300 \times 270 \times 16) \times 10^{-3} = 583 \text{kN} \gg V^* = 94 \text{kN} \)
- \( \Phi V_{ep_h} = 0.9 \times (0.5 \times 300 \times 150 \times 16) \times 10^{-3} = 324 \text{kN} \gg 2F^* = 220 \text{kN} \)

Endplate is satisfactory in shear.
Note that similar checks for endplate in bearing and tearing as per Equation A-10 are also required, although these are not critical.

Figure A-3: Design of endplate for flexible T-stub
Appendix B

Strength design procedure for channel side plate connection with Ajax ONESIDE

A quantitative procedure for designing channel side plate connection for hollow section columns using Ajax ONESIDE blind bolts is outlined in this appendix. The design criteria for this connection are as follow:

1) The connection deformation should satisfy the allowable structural drift limits.

2) Failure of blind bolts and standard bolts before the channels, side plates, column face or the beam reach its plastic capacity and experience strain hardening, should be prevented.

3) Channel section thickness should be at least equal to column thickness.
4) Minimum side plate thickness is 10mm.

**B.1 Design Steps**

In this section, the design procedure of a channel side plate connection for 310UB32 beam and 150×150×6 SHS column is presented. The same case study is used as per Appendix A, replacing the T-stub connection with channel side plate connection. Assume a grid of 6m × 4m for a residential or light commercial structure.

**Design Actions**

Superimposed dead load + self weight = 1.5kPa

Live load = 4kPa

Shear force, \( V^* = \frac{(1.2 \times 1.5 + 1.5 \times 4) \times 4 \times 6}{2} = 94kN \)

Design connection to have moment capacity that matches the lower of the beam or column capacity, hence \( M^* = 55kNm \) (as per column’s capacity)

**Step 1: Determine design force in channel face, \( F^* \)**

\[
F^* = \frac{M^*}{d_{bm}} \quad \text{(as per Eq. A-7)}
\]

\[
F^* = \frac{55}{0.32} = 172kN
\]

**Step 2: Check shear capacity in channel section**

\[
\Phi V_{channel} > F^* \quad \text{(B-1)}
\]

where

\[
\Phi V_{channel} = 0.6f_y A_v
\]
\[ 0.6 \times 300 \times (150 \times 6) \times 10^{-3} = 162 \text{kN} < F^*, \text{ hence requires channel next size up} \]

Channel size of 9mm, \( \phi V_{\text{channel}} = 243 \text{kN} \)

**Step 3:** Determine minimum number of standard bolts, \( N_{sb} \) to transfer shear force in the channel face, check for bearing of ply

There are 2 sets of standard bolts:

1st set of standard bolts connect beam flange to channel face. These bolts are subjected to shear force of \( F^* \).

2nd set of standard bolts connect channel legs to side plates. These bolts are subjected to combined vertical shear, \( V^*/2 \) and horizontal shear \( F^* \). Hence 2nd set of standard bolts are more critical.

Total shear on 2nd set of standard bolts (connecting channel legs to side plates).

\[
V^\text{total}_* = \sqrt{(V^*/2)^2 + (F^*)^2} = \sqrt{(94/2)^2 + (172)^2} = 178 \text{kN}
\]
Use Equation A-7 to determine shear capacity of a standard 8.8 M16 bolt

\[ \Phi V_f = 0.8(0.62)k_r f_u (n_A c + n_A o) \]  

(as per Eq. A-7)

\[ \Phi V_{f_{sb}} = 0.8 \times 0.62 \times 1.0 \times 830 \times (1 \times 144) \times 10^{-3} \]

= 59kN per M16 standard bolt

\[ N_{sb} = \frac{V_{total}^*}{\Phi V_{f_{sb}}} = \frac{178}{59} = 3 \] , therefore adopt \( N_{sb} = 4 \) bolts

Hence 4 bolts for 1st set connecting beam flange to channel face, and 4 bolts for 2nd set connecting channel legs to side plates (for top and bottom connection)

Check ply in bearing and tearing

\[ \Phi V_b = \text{less of } 0.9a_e t_p f_{up} \]  

and \( 0.9(3.2)d_b t_p f_{up} \)  

(as per Eq. A-10)

where

\( a_e \) = edge distance

\( t_p \) = ply thickness, channel is 9mm thick while side plate is 10mm, hence channel is more critical

\( d_b \) = diameter of the bolt, use M16 blind bolts

\[ 0.9a_e t_p f_{up} = 0.9 \times 30 \times 9 \times 440 \times 10^{-3} = 107kN \]

\[ 0.9(3.2)d_b t_p f_{up} = 0.9 \times 3.2 \times 16 \times 9 \times 440 \times 10^{-3} = 182kN \]

Hence \( \Phi V_{b_{sb}} = 107kN \) per bolt which satisfies \( V_{total^*}/\text{bolt} \) of 45kN.

**Step 4:** Determine minimum number of blind bolts, \( N_{bb} \) required to carry tension force from beam flange to column side walls, check for bearing of ply

Bolt shear capacity,

\[ \Phi V_f = 0.8(0.62)k_r f_u (n_A c + n_A o) \]  

(as per Eq. A-7)

Assume threads are included; sleeve of blind bolt will contribute to \( A_c \)
Outer diameter of sleeve = 22.5mm, inner diameter = 16.5mm

\[ \Phi V_{f-bb} = 0.8 \times 0.62 \times 1.0 \times 830 \times (1 \times 327) \times 10^{-3} \]
\[ = 134 \text{kN per M16 blind bolt} \]

Blind bolts at the top or bottom are subjected to vertical shear, \( V^*/2 \) and horizontal shear \( F^* \) (refer Figure B-1)

Total shear on blind bolts at the top or bottom,
\[ V_{\text{total}}^* = \sqrt{(V^*/2)^2 + (F^*)^2} \]
\[ = 178 \text{kN} \quad \text{(as per Step 3)} \]

\[ N_{bb} = \frac{V_{\text{total}}^*}{\Phi V_{f-bb}} = \frac{178}{134} = 1.3 \]

\[ \therefore \text{Provide 2 blind bolts on each face of tube side wall, hence total of 4 blind bolts provided at the top and another 4 blind bolts at the bottom, each blind bolt has ample capacity in shear} \]

Check ply in bearing and tearing
\[ \Phi V_b = \text{lesser of } 0.9a_e t_p f_{up} \quad \text{as per Eq. A-10} \]
\[ \text{and } 0.9(3.2)d_b t_p f_{up} \]

where
\[ a_e = \text{edge distance} \]
\[ t_p = \text{ply thickness, tube is 6mm thick while side plate is 10mm, hence tube is more critical} \]
\[ d_b = \text{diameter of the bolt, use M16 blind bolts} \]

\[ 0.9a_e t_p f_{up} = 0.9 \times 40 \times 6 \times 430 \times 10^{-3} = 93 \text{kN} \]
\[ 0.9(3.2)d_b t_p f_{up} = 0.9 \times 3.2 \times 16 \times 6 \times 430 \times 10^{-3} = 119 \text{kN} \]

Hence \( \Phi V_{b,bb} = 93 \text{kN per bolt which satisfies which satisfies } V_{\text{total}/\text{bolt}} \text{ of 45kN} \).
**Step 5:** Check side plates in tension and shear

Plate in tension

\[ \Phi N_t = \text{lesser of } 0.9A_g f_y \quad \text{(as per Eq. A-4)} \]
and \[0.9(0.85)k_t A_n f_u\]

\[0.9A_g f_y = 0.9 \times 90 \times 10 \times 300 \times 10^{-3} = 243 \text{kN}\]
\[0.9(0.85)k_t A_n f_u = 0.9 \times 0.85 \times (90 \times 10 - 10 \times 24 \times 1) \times 440 \times 10^{-3} = 222 \text{kN}\]

Hence \( \Phi N_t = 222 \text{kN} > F^* = 178 \text{kN} \), hence 10mm side plate is satisfactory in tension.

Plate in shear

\[ \Phi V_{plate} > V/2 \quad \text{(B-2)} \]

where

\[ \Phi V_{channel} = 0.6 f_y A_v \]
\[ = 0.6 \times 300 \times (90 \times 10) \times 10^{-3} \]
\[ = 162 \text{kN} > V/2 = 47 \text{kN} \), hence 10mm side plate is satisfactory.
Appendix C

Strength design procedure for blind bolted simple connections with Ajax ONESIDE

A quantitative procedure to design double angle cleats or flexible endplate connection for hollow section columns using Ajax ONESIDE blind bolts is outlined in this appendix based on BCSA/SCI: Design of Joints in Simple Construction (2002). This is intended for connection design in the secondary direction of framing. The design criteria for the double angle cleats and flexible endplate connections based on the BCSA/SCA rules (refer Figure C-1) are:

1) Cleats or endplate should be positioned as close to the top flange as possible to provide adequate positional restraint.

2) Length of the angle cleats or endplate, \( l \) should be at least 0.6 times beam depth, \( D \).
3) Cleats or endplate thickness, \( t_p \) must be at least 8mm.

**Double Angle Cleats**

- End projection \( t_p \), approx 10mm
- Length of cleat \( l \geq 0.8D \)
- Face of column

**Flexible End Plates**

- Length of plate \( l \geq 0.8D \)
- Face of column

![Diagram of Double Angle Cleats and Flexible End Plates](image)

*Figure C-1: Design criterion for simple blind bolted connections (British Steel, 2000)*

Assume secondary beams of 4m length and 2 m spacing with double angle cleat connections for the 3 storey, 6m × 4m grid frame discussed in Chapter 8, 180UB16 beams are required to satisfy strength and deflection limits under gravity loading in the secondary direction.

**Design Actions**

- Superimposed dead load + self weight = 1.5kPa
- Live load = 4kPa
- Shear force, \( V^* = \frac{[(1.2 \times 1.5 + 1.5 \times 4) \times 2] \times 4}{2} = 31kN \)
Step 1: Determine number of bolts to carry shear in connection

As per the calculations in Appendix A,
Shear capacity for a single M16 ONESIDE blind bolt, \( \phi V_{f-bb} = 134 \text{kN} \) for single shear plane.

Hence number of blind bolts required, \( N_{bb} = \frac{V_*}{\phi V_{f-bb}} < 1 \), adopt min number of two M16 blind bolts per angle.

Shear capacity for a standard 8.8 bolt, \( \phi V_{f-sb} = 59 \times 2 = 118 \text{kN} \) for double shear plane.

Hence number of standard M16 bolts required, \( N_{sb} = \frac{V_*}{\phi V_{f-sb}} < 1 \), hence adopt two M16 standard bolts tying the legs of the double angles to the beam web.

Step 2: Check for local shear and bearing capacity of the SHS column wall and bolts (refer Figure C-2)

![Critical sections](image)

Figure C-2: Shear and bearing capacity check
From Step 1, \( n \) (number of bolt rows) = 2

(i) Shear check

\[
\frac{Q}{2} \leq P_v \tag{C-1}
\]

where

\[
P_v = \text{local shear capacity of the SHS column wall, lesser of } 0.6f_{y,c}A_v \text{ and } 0.5f_{u,c}A_{v,\text{net}}
\]

and

\[
e_t = \frac{g}{2} + (n-1)p, \text{ where} \]

\[
g = \text{bolt gauge width, 70mm} \]

\[
n = \text{number of bolt rows, 2} \]

\[
p = \text{bolt pitch, 60mm} \]

\[
\therefore e_t = \frac{70}{2} + (2-1)\times60 = 95\text{mm}
\]

\[
A_v = \text{area in shear} \]

\[
= e_t t_c \text{ with } e_t \leq 5d_h, \]

\[
d_h = \text{bolt hole diameter, 24mm for M16 blind bolt} \]

\[
t_c = \text{column wall thickness, 6mm} \]

\[
\therefore A_v = 95 \times 5 = 475\text{mm}^2
\]

\[
A_{v,\text{net}} = A_v - n d_h t_c
\]

\[
\therefore A_v = 475 - 2 \times 24 \times 6 = 187\text{mm}^2
\]

\[
0.6f_{y,c}A_v = 0.6 \times 350 \times 475 \times 10^{-3} = 100\text{kN}
\]

\[
0.5f_{u,c}A_{v,\text{net}} = 0.5 \times 430 \times 187 \times 10^{-3} = 40\text{kN}
\]

\[
\therefore P_v = 40\text{kN}
\]

\[
Q/2 = V/2 = 31\text{kN}/2 = 16\text{kN}
\]

\( P_v > Q/2 \), hence column is satisfactory in shear.
(ii) Bearing check

\[
\frac{Q}{2} \leq P_{bsc} \quad \text{(C-2)}
\]

where

\[ P_{bsc} = \text{bearing capacity of SHS column wall} \]

\[ = d_h t_c f_{u,c} \]

\[ \therefore P_{bsc} = 24 \times 6 \times 430 \times 10^{-3} = 62 \text{kN} > Q/2 = 16 \text{kN} \quad \text{(satisfactory in bearing)} \]

**Step 3: Check for bolt pull-out capacity (refer Figure C-3)**

Design for a minimum tie force of 75kN for structural integrity

Tie force \( \leq \Sigma \text{Bolt pull out resistance, } F_{ps} \) as is given by Equation A-3 in Appendix A

\[ F_{ps} = 0.6 f_{c,y} \pi t_c d_b = 63 \text{kN per M16 blind bolt,} \]

4 blind bolts in total, hence \( \Sigma F_{ps} = 63 \times 4 = 252 \text{kN} > 75 \text{kN} \) of tie force requirement, therefore tube is satisfactory for bolt pull-out.
Step 4: Check for column tube face yielding against minimum tie force for structural integrity (refer Figure C-4)

![Figure C-4: Yielding of column tube face](image)

Tie force $\leq$ Column face plastification load, $N_{pl}$ as is given by Equation A-2 in Appendix A

$$N_{pl} = \frac{f_{c,y} t_c}{2} \left[ \frac{2(h_b - d_r)}{b'} + 4 \sqrt{\frac{1 - \frac{c}{b'}}{1 - \frac{c}{b'}}} \right]$$

$$= 93 \text{kN} > \text{Tie force} = 75 \text{kN}, \text{ hence column face is satisfactory.}$$
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