In presenting this thesis in partial fulfilment of the requirements for an advanced degree at the University of British Columbia, I agree that the Library shall make it freely available for reference and study. I further agree that permission for extensive copying of this thesis for scholarly purposes may be granted by the head of my department or by his or her representatives. It is understood that copying or publication of this thesis for financial gain shall not be allowed without my written permission.

Department of **Civil Engineering**

The University of British Columbia  
Vancouver, Canada

Date *August 21, 2000*
ABSTRACT

Braced timber frames are efficient lateral load resistance systems in buildings where large open spaces are required, and the more commonly used timber shear wall systems cannot be utilized. Braced timber frames allow for flexibility in the design and use the wood in its strongest direction – parallel to grain in tension or compression. For application in high-risk earthquake zones, however, the ductility of the system is a concern, since energy absorption is typically limited to the connection region. This study focused on seismic behaviour of braced timber frames with particular emphasis on investigating the influence of different connection details on the overall stiffness, strength and seismic energy absorption capacity of the frame.

Monotonic tension and cyclic quasi-static tests were conducted on a variety of connections typically used in braced timber frames, utilizing different diameter bolts and high strength glulam rivets with steel side plates. Shake table tests were subsequently conducted on a selected number of single storey braced frames with some of the connections previously tested and on a two storey braced timber frame model with riveted connections. The experimental results from quasi-static tests and shake table tests were used to establish and verify non-linear analytical models representing the load-deformation behaviour of different connections. These hysteresis curves were then introduced in analytical braced frame models. These models were used in a number of non-linear static and dynamic analyses to determine the response of braced frames to
the input of five different records from previous earthquakes. From these analyses it was possible to determine the influence of different connection details on the seismic response of the selected types of braced timber frames. Based on the results from the analytical part of the study, an estimate was made on the appropriate force modification factors (R-factors) for earthquake resistant design of braced timber frames, as used in the National Building Code of Canada. Finally, some design and construction recommendations are discussed to inform the reader of the details required to obtain an adequate seismic performance. Possible ways of improving the seismic behaviour of braced timber frames are presented as well.
TABLE OF CONTENTS

ABSTRACT ......................................................................................................................... ii
LIST OF TABLES ................................................................................................................. x
LIST OF FIGURES ............................................................................................................. xii
NOTATION AND ABBREVIATIONS .................................................................................. xix
ACKNOWLEDGEMENTS ..................................................................................................... xxi
DEDICATION ..................................................................................................................... xxiii
FOREWORD ...................................................................................................................... xxiv

CHAPTER 1 INTRODUCTION .............................................................................................. 1
  1.1 PROBLEM OVERVIEW ............................................................................................. 1
  1.2 RESEARCH OBJECTIVES ....................................................................................... 4
  1.3 SCOPE ....................................................................................................................... 4
  1.4 THESIS OUTLINE .................................................................................................... 6

CHAPTER 2 LITERATURE REVIEW .................................................................................... 8
  2.1 BRIEF HISTORY OF TIMBER STRUCTURES .......................................................... 8
  2.2 TIMBER AS A STRUCTURAL MATERIAL ................................................................. 13
  2.3 PAST SEISMIC PERFORMANCE OF TIMBER BUILDINGS .................................... 16
  2.4 SEISMIC BEHAVIOUR OF TIMBER STRUCTURES ............................................... 19
  2.5 CONNECTORS FOR TIMBER STRUCTURES IN SEISMIC REGIONS ................. 23
3.3.2.2 Connection Properties ................................................. 74
3.3.2.3 Energy Dissipation .................................................... 77
3.3.2.4 Failure Modes ......................................................... 79
3.4 SUMMARY ........................................................................ 81

CHAPTER 4 SINGLE BRACE SHAKE TABLE TESTS .......................... 82
4.1 OBJECTIVES AND SCOPE ................................................. 82
4.2 DESCRIPTION OF THE TESTING FACILITY ......................... 83
4.3 SHAKE TABLE TEST SETUP .............................................. 85
4.4 SPECIMEN CONFIGURATION ............................................. 90
4.5 INSTRUMENTATION AND DATA ACQUISITION ..................... 91
4.6 TESTING PROCEDURES .................................................... 94
  4.6.1 Impact Hammer Tests .................................................. 94
  4.6.2 Choice of Excitation Record for Shake Table Tests .......... 94
  4.6.3 Testing Sequence ...................................................... 96
4.7 MATERIAL PROPERTIES .................................................. 98
4.8 RESULTS AND DISCUSSION .............................................. 100
  4.8.1 Impact Hammer Tests .................................................. 100
  4.8.2 Shake Table Tests ..................................................... 101
    4.8.2.1 Data Pre-Processing ............................................. 101
    4.8.2.2 Time History Response Parameters ......................... 102
    4.8.2.3 Load-Deformation Relationships ............................. 107
    4.8.2.4 Frequency Domain Analyses .................................. 113
    4.8.2.5 Energy Dissipation ............................................. 116
4.8.2.6 Failure Characteristics ........................................ 118

4.9 SUMMARY .................................................................... 119

CHAPTER 5 ANALYTICAL PREDICTION OF SINGLE BRACE SHAKE TABLE TESTS .................................................. 120

5.1 OBJECTIVES ............................................................... 120

5.2 ANALYTICAL MODEL DEVELOPMENT ................................. 121

5.2.1 3-D Linear Elastic Model ........................................... 122

5.2.2 2-D Non-Linear Analytical Model ................................. 123

5.3 RESULTS AND DISCUSSION ........................................... 127

5.3.1 Modal Analysis ......................................................... 127

5.3.2 Time History Dynamic Analysis .................................... 130

5.3.2.1 Linear Analysis ................................................... 130

5.3.2.2 Non-Linear Analysis ............................................. 132

5.4 SUMMARY .................................................................... 136

CHAPTER 6 FORCE MODIFICATION FACTORS FOR BRACED TIMBER FRAMES .................................................. 138

6.1 OBJECTIVES ............................................................... 138

6.2 DEVELOPMENT OF THE ANALYTICAL MODELS .............. 139

6.3 EARTHQUAKE GROUND MOTIONS .................................. 146

6.4 NON-LINEAR TIME HISTORY DYNAMIC ANALYSES .......... 150

6.4.1 Analysis Procedure .................................................. 151

6.4.2 Fundamental Period .................................................. 154

6.4.3 Non-Linear Response Parameters .................................. 155

6.5 FORCE MODIFICATION FACTORS ................................. 156

6.5.1 Influence of Frame Geometry on R-Factors ...................... 162
<table>
<thead>
<tr>
<th>Chapter</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.6</td>
<td>SUMMARY</td>
<td>235</td>
</tr>
<tr>
<td>CHAPTER 8</td>
<td>ANALYTICAL PREDICTION OF THE MODEL RESPONSE</td>
<td>237</td>
</tr>
<tr>
<td>8.1</td>
<td>ANALYTICAL MODELS</td>
<td>237</td>
</tr>
<tr>
<td>8.1.1</td>
<td>3-D Linear Elastic Model</td>
<td>238</td>
</tr>
<tr>
<td>8.1.2</td>
<td>2-D Non-Linear Model</td>
<td>239</td>
</tr>
<tr>
<td>8.2</td>
<td>RESULTS AND DISCUSSION</td>
<td>241</td>
</tr>
<tr>
<td>8.2.1</td>
<td>Modal Analysis</td>
<td>241</td>
</tr>
<tr>
<td>8.2.2</td>
<td>Time History Dynamic Analysis</td>
<td>243</td>
</tr>
<tr>
<td>8.3</td>
<td>SUMMARY</td>
<td>249</td>
</tr>
<tr>
<td>CHAPTER 9</td>
<td>SUMMARY, CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH</td>
<td>251</td>
</tr>
<tr>
<td>9.1</td>
<td>SUMMARY</td>
<td>251</td>
</tr>
<tr>
<td>9.2</td>
<td>CONCLUSIONS AND DESIGN RECOMMENDATIONS</td>
<td>253</td>
</tr>
<tr>
<td>9.3</td>
<td>RECOMMENDATIONS FOR FUTURE RESEARCH</td>
<td>259</td>
</tr>
<tr>
<td>CHAPTER 10</td>
<td>BIBLIOGRAPHY</td>
<td>262</td>
</tr>
<tr>
<td>APPENDIX A</td>
<td></td>
<td>280</td>
</tr>
</tbody>
</table>
LIST OF TABLES

Table 3.1. Configuration and specifications for the tested connections ........................................ 59
Table. 3.2. Average connection properties obtained from monotonic tension tests .................. 68
Table 3.3. Average connection properties from cyclic tests based on first-cycle envelope curve ................................................................................................................. 74
Table 3.4. Average connection properties from cyclic tests based on stabilised envelope curve ......................................................................................................................... 75
Table 4.1. List of the instruments used and their location ............................................................. 93
Table 4.2. A summary of shake table tests performed ................................................................. 97
Table 4.3. Connection properties for weaker connections from shake table tests ....................... 111
Table 5.1. Final parameters used to model different braces using the Florence model ............ 127
Table 5.2. Fundamental frequencies of the models with different connections ....................... 129
Table 6.1. Main characteristics of the acceleration records used in the analyses ..................... 146
Table 6.2. Main characteristics of the larger set of records used for preliminary analyses ......... 147
Table 6.3. Fundamental periods of braced frame model for different design configurations ................................................................. 154
Table 6.4. Maximum R-factors for braced timber frames with different connections ............... 160
Table 6.5. Basic geometry information for the frame models used in analyses ....................... 163
Table 6.6. Fundamental periods for frame models with different aspect ratios ....................... 164
Table 6.7. Maximum R-factors for frames with different aspect ratios .................................... 166
Table 6.8. Storey mass distribution for all three models in use ................................................... 172
Table 6.9. Maximum R-factors for multi-storey braced frames with different connections ....... 174
Table 7.1. List of the sensors used during the shake table tests and their location .................... 184
Table 7.2. List of the accelerometers used during the impact hammer tests and their location ................................................................................................. 186
List of Tables

Table 7.3. A summary of shake table tests performed .......................................................... 191
Table 7.4. Frequency and damping values for the fundamental mode of the model .......... 194
Table 7.5. Maximum accelerations recorded at each storey of the model ......................... 203
Table 7.6. List of the sensors used during the pushover test and their location .................. 231
Table 8.1. Initial dynamic characteristics of both analytical models ............................... 242
LIST OF FIGURES

Figure 2.1. Typical Chinese Building system of 13th century .................................................. 9
Figure 2.2. Box frame timber system used in medieval Europe; a) Tie beam, b) wall plate, c) wall post, d) bressummer, e) storey plate, f) sole plate, g) summer, h) girder ........................................................................................................ 10
Figure 2.3. Stress-strain relationships for wood (Buchanan, 1994) .............................................. 14
Figure 2.4. Typical hysteretic behaviour of a timber structure component ................................ 21
Figure 2.5. Formation of a cavity around the nail during cyclic loading and the pullout effect ........................................................................................................................................ 27
Figure 2.6. Typical hysteresis loops of a dowel connection at different load level (Ceccotti 1996) ................................................................................................................................. 28
Figure 2.7. Typical hysteretic behaviour of connections with a) slender and b) stocky bolts .................................................................................................................................................. 30
Figure 2.8. Failure modes assumed in European Yield Model for three-member joint ...... 31
Figure 2.9. Typical column to beam riveted connection .............................................................. 33
Figure 2.10. Braced timber frame in residential construction in Japan ....................................... 35
Figure 2.11. Different types of braced timber frames ................................................................. 36
Figure 2.12. Considerations of braced timber frames ................................................................. 38
Figure 2.13. Mixed lateral load resistant systems ....................................................................... 39
Figure 2.14. Hysteresis loops for diagonally braced frame (dean et al., 1989) ...................... 40
Figure 2.15. Typical hysteresis loop obtained during the testing (Yasamura, 1990) .............. 41
Figure 2.16. Seismic response factor S in the 1995 version of NBCC ........................................ 46
Figure 2.17. Force-deformation relationship for simple elastic-perfectly plastic systems .. 48
Figure 2.18. Influence of structural period on ductile force reduction (Priestley, 1992) .... 49
Figure 2.19. Behaviour factor as a function of the predominant frequency (Ceccotti et al. 1988) ................................................................. 53
Figure 3.1. Typical details of riveted and bolted braced frame connections .................. 57
Figure 3.2. A glulam rivet and steel plate hole arrangement of a riveted connection ........ 58
Figure 3.3. Quasi-static test setup: a) simplified scheme; b) picture of the setup .......... 61
Figure 3.4. Definition of yield and ultimate displacements according to different standards .......................................................... 63
Figure 3.5. Cyclic testing protocol used for quasi-static connection test ...................... 64
Figure 3.6. Relative wood densities of the wood specimen ........................................ 65
Figure 3.7. Cumulative distribution curve for MOE .................................................. 66
Figure 3.8. Load-displacement curves from tension tests on 12.7 mm bolts ................. 67
Figure 3.9. Load-deformation curves from monotonic tension tests of the first test group . 69
Figure 3.10. a-b) Hysteretic curves from cyclic tests of the connections from the first test group ........................................................................ 70
Figure 3.10. c-d) Hysteretic curves from cyclic tests of the connections from the first test group ........................................................................ 71
Figure 3.11. Typical hysteretic curves from cyclic tests of the connections from the second test group ................................................................. 72
Figure 3.12. Stabilised hysteresis loops of the bolted and riveted connection at different deformation levels .......................................................... 73
Figure 3.13. Different envelope curves for connections from the first test group .......... 76
Figure 3.14. Dissipated energy during the cyclic tests for all specimens from the first test group ................................................................. 78
Figure 3.15. Typical riveted connection failure mode .................................................. 79
Figure 3.16. Glulam member from a bolted connection after cyclic testing a) cross section; b) side view ................................................................. 80
Figure 4.1. Simplified scheme of the tests setup for the shake table tests of single braced frames ....................................................................... 85
Figure 4.2. Single brace shake table setup ................................................................... 86
Figure 4.3. A more detailed drawing of the four-hinged frame - front view .................. 87
Figure 4.4. A more detailed drawing of the four-hinged frame – side view ................. 88
Figure 4.5. Connecting the wood brace member to a) top and b) bottom of the testing frame 89
Figure 4.6. Location of the instrumented points on the frame 92
Figure 4.7. Joshua Tree acceleration record; a) time history; b) PSA response spectrum 95
Figure 4.8. Test setup for determining the MOE of the glulam beams 99
Figure 4.9. Typical amplitude spectrum of the system response during the impact hammer test 101
Figure 4.10a. Time histories of selected quantities obtained from test number 6 -12.7 mm bolts 102
Figure 4.10b. Time histories of selected quantities obtained from test number 6 -12.7 mm bolts 103
Figure 4.11. Time histories of selected quantities obtained from test number 8 -9.5 mm bolts 104
Figure 4.12. Time histories of selected quantities obtained from test number 9 -9.5 mm bolts + rods 104
Figure 4.13. Time histories of selected quantities obtained from test number 10 –Rivets 105
Figure 4.14. Time histories of selected quantities obtained from test number 12 –19 mm bolts 105
Figure 4.15a. Selected load deformation relationships for both braced connections 108
Figure 4.15b. Selected load deformation relationships for both braced connections 109
Figure 4.16. Shake table tests envelopes for different connections 111
Figure 4.17a. Fourier amplitude spectra of frames with different connections 113
Figure 4.17b. Fourier amplitude spectra of frames with different connections 114
Figure 4.18. Dynamic amplification factor in frequency domain for frame with 19 mm bolts 115
Figure 4.19. Energy dissipation in different brace connections during shake table tests 116
Figure 4.20. Dissipated energy in connections during cyclic and shake table test 117
Figure 4.21. Typical failure modes in bolted and riveted connection during shake table tests 118
Figure 5.1. A 3-D linear elastic model of single braced frames in SAP90 122
Figure 5.2. Mathematical model for connection behavior used in the analyses 124
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.3</td>
<td>A 2-D non-linear DRAIN-2DX model of single braced frames</td>
<td>125</td>
</tr>
<tr>
<td>5.4</td>
<td>Fundamental mode shape obtained from the linear elastic 3-D model (SAP90)</td>
<td>128</td>
</tr>
<tr>
<td>5.5</td>
<td>First natural mode of the analytical non-linear 2-D model (DRAIN-2DX)</td>
<td>129</td>
</tr>
<tr>
<td>5.6</td>
<td>A comparison of an acceleration history measured and obtained from SAP90 model</td>
<td>131</td>
</tr>
<tr>
<td>5.7</td>
<td>A comparison of relative displacement measured and obtained from SAP90 model</td>
<td>132</td>
</tr>
<tr>
<td>5.8</td>
<td>A comparison of acceleration histories measured and obtained from DRAIN models</td>
<td>133</td>
</tr>
<tr>
<td>5.9</td>
<td>Load-deformation relationships of braces with different connections compared to the corresponding shake table test envelope curves</td>
<td>134</td>
</tr>
<tr>
<td>5.10</td>
<td>A comparison of the dissipated energies for braces with four different connections</td>
<td>135</td>
</tr>
<tr>
<td>6.1</td>
<td>Elevation of braced timber frame of a typical industrial type building</td>
<td>139</td>
</tr>
<tr>
<td>6.2</td>
<td>Concentrically trussed braced timber frame as a basic lateral load resistant system</td>
<td>140</td>
</tr>
<tr>
<td>6.3</td>
<td>Elevation of a typical commercial type of braced frame building used in analysis</td>
<td>141</td>
</tr>
<tr>
<td>6.4</td>
<td>Non-linear mathematical models in DRAIN-2DX used in analyses</td>
<td>142</td>
</tr>
<tr>
<td>6.5</td>
<td>Tested and analytically predicted hysteresis behaviour of a riveted connection</td>
<td>144</td>
</tr>
<tr>
<td>6.6</td>
<td>Defining the analytical model for overall brace behaviour</td>
<td>145</td>
</tr>
<tr>
<td>6.7</td>
<td>Acceleration histories of the records used in the analyses</td>
<td>148</td>
</tr>
<tr>
<td>6.8a</td>
<td>Pseudo acceleration response spectra for the records used in the analyses</td>
<td>149</td>
</tr>
<tr>
<td>6.8b</td>
<td>Pseudo acceleration response spectra for the records used in the analyses</td>
<td>150</td>
</tr>
<tr>
<td>6.9</td>
<td>Single storey braced timber frame model analysed</td>
<td>152</td>
</tr>
<tr>
<td>6.10</td>
<td>Fundamental mode shape of the single storey analytical model</td>
<td>154</td>
</tr>
<tr>
<td>6.11</td>
<td>Selected time histories of the response at the top of the braced frame model</td>
<td>155</td>
</tr>
<tr>
<td>6.12</td>
<td>Load-deformation response of the bottom brace for frame with riveted connections</td>
<td>156</td>
</tr>
<tr>
<td>6.13</td>
<td>Deformation demands of braced timber frames with 12.7 mm bolted connections</td>
<td>157</td>
</tr>
</tbody>
</table>
Figure 6.14. Deformation demands of braced timber frames with 9.5 mm bolted connections ................................................................. 158
Figure 6.15. Deformation demands of braced timber frames with riveted connections ..... 159
Figure 6.16. Deformation demands of a braced timber frame with 19 mm bolted connections .................................................................................. 160
Figure 6.17. Brace demands as a function of $W/F_d$ ratio for frames with different connections .............................................................................. 162
Figure 6.18. Deformation demand for frames with different aspect ratios for the VAN-68 record ........................................................................................................ 165
Figure 6.19. Cyclic pushover analysis using the basic braced frame model .................... 167
Figure 6.20. Period dependency of ductility demand for selected braced timber frames ... 169
Figure 6.21. Period dependency of average demand for frames with different connections 170
Figure 6.22. Storey mass distribution for all three model cases used ................................. 172
Figure 6.23. First, second and a third mode shape of typical three storey model – Case I .. 173
Figure 6.24. Deformation demand of multi-storey frames with bolted and riveted connections ........................................................................................................ 174
Figure 6.25. Deformation demands of single and multi-storey frames with riveted connections ........................................................................................................ 175
Figure 7.1. Braced frame model ready for the shake table tests (NE view) ....................... 178
Figure 7.2. 3-D drawing of the braced frame model .......................................................... 179
Figure 7.3. Location of the sensors on the south frame of the braced frame model .......... 182
Figure 7.4. Location of the sensors on the north frame of the brace model ....................... 183
Figure 7.5. Normalized accelerograms of the records used for the shake table tests .......... 187
Figure 7.6. Pseudoacceleration response spectra of the records used for the tests .......... 188
Figure 7.7. Acceleration signals at the top of the model during 3X and 3Y impact tests ... 192
Figure 7.8. Spectral amplitude of the top accelerations from impact tests 3X and 3Y ...... 193
Figure 7.9a. Acceleration at the top of the model for various levels of VAN-29 input ...... 197
Figure 7.9b. Acceleration at the top of the model for various levels of VAN-29 input ...... 198
Figure 7.10a. Acceleration at the top of the model for various levels of Watsonville input 198
Figure 7.10b. Acceleration at the top of the model for various levels of Watsonville input 199
Figure 7.11a. Acceleration at the top of the model for various levels of Joshua Tree input 199
Figure 7.11b. Acceleration at the top of the model for various levels of Joshua Tree input 200
Figure 7.12. Acceleration along the height of the model for test 7 – VAN-29 (50%) ........ 201
Figure 7.13. Acceleration along the height of the model for test 13 – VAN-29 (100%) .... 201
Figure 7.14. Acceleration along the height of the model for test 15 – VAN-29 (100%) .... 202
Figure 7.15. Displacements at the top of the north frame for various levels of VAN-29 input ................................................................................................................................. 204
Figure 7.16a. Displacements at the top of the north frame for various levels of Watsonville input ......................................................................................................................... 204
Figure 7.16b. Displacements at the top of the north frame for various levels of Watsonville input ................................................................................................................................. 205
Figure 7.17. Displacement at the top of the north frame for 100% Joshua Tree input ....... 206
Figure 7.18. Displacement difference at the top of the north and south frame for different tests ................................................................................................................................. 207
Figure 7.19. Storey drifts recorded during test 14 north frame – Watsonville 100% ....... 208
Figure 7.20. Storey drifts recorded during test 15 north frame – VAN-29 100% ........... 208
Figure 7.21. Deformations of the upper brace of the north frame during the last six tests .. 210
Figure 7.22. Deformations of the lower brace of the north frame during the last six tests .. 211
Figure 7.23. Deformations of the upper brace of the south frame during the last six tests .. 212
Figure 7.24. Deformations of the lower brace of the south frame during the last six tests .. 213
Figure 7.25. Deformations in both connections of the upper brace of the south frame – tests 14 ................................................................................................................................. 214
Figure 7.26. Deformations in both connections of the lower brace of the south frame – tests 14 ................................................................................................................................. 214
Figure 7.27. Load-deformation relationships of the bottom brace – north frame .......... 216
Figure 7.28. Load-deformation relationships of the top brace – north frame ............... 217
Figure 7.29. Load-deformation relationships of the bottom brace – north frame .......... 218
Figure 7.30. Load-deformation relationships of the top brace – south frame ............... 219
Figure 7.31. Load-deformation relationships of the bottom brace – south frame .......... 220
Figure 7.32. a) Fourier amplitude spectra of the three input motions used and b) the spectra of top accelerations obtained during the corresponding tests ................. 223
Figure 7.33. The amplitude ratio of input and output spectrum for test 1 – Joshua Tree
List of Figures

Figure 7.34. Fourier amplitude spectra for six tests from 10% to 100% - Watsonville earthquake ............................................................ 224

Figure 7.35. Shapes of the vibration of the model for different earthquakes ................................................. 227

Figure 7.37. A picture of the braced frame model ready for the pushover test ........................................ 229

Figure 7.38. A 3-D drawing of the braced frame model ready for pushover test ........................................ 229

Figure 7.39. Braced frame model at point of maximum deformation during the pushover test ................................................................. 232

Figure 7.40. Load-displacement relationship for the entire braced frame model .......................... 233

Figure 7.41. Load-deformation relationships for the four different braces of the model ... 234

Figure 7.42. Typical failure of riveted connection during the pushover test ................................. 235

Figure 8.1. A 3-D linear elastic model of braced timber frame (SAP90) .................................................. 238

Figure 8.2. A 2-D non-linear DRAIN-2DX model of the braced frame ............................................. 240

Figure 8.3. First two mode shapes obtained from the linear elastic 3-D model (SAP90) ................................................................. 241

Figure 8.4. First two mode shapes of the non-linear 2-D model (DRAIN-2DX) .......... 242

Figure 8.5. A comparison between the measured and analytically obtained deformations and accelerations at the top of the north frame for test number 11 .............. 244

Figure 8.6. A comparison between the measured and analytically obtained hysteresis loops in the braces of the north frame during test number 11 ......................... 246

Figure 8.7. A comparison between the measured and analytically obtained deformations and accelerations at the top of the north frame for test number 12 ................. 248

Figure 8.8. A comparison between the measured and analytically obtained hysteresis loops in the braces of the north frame during test number 12 ......................... 249
<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-D</td>
<td>Two Dimensional</td>
</tr>
<tr>
<td>3 DOF</td>
<td>Three Degrees of Freedom</td>
</tr>
<tr>
<td>3-D</td>
<td>Three Dimensional</td>
</tr>
<tr>
<td>ASTM</td>
<td>American Society for Testing and Materials</td>
</tr>
<tr>
<td>CEN</td>
<td>Comité Européen de Normalisation</td>
</tr>
<tr>
<td>CSA</td>
<td>Canadian Standards Association</td>
</tr>
<tr>
<td>CSA-O86.1-94</td>
<td>Engineering Design in Wood (Limit States)</td>
</tr>
<tr>
<td>CSMIP</td>
<td>California Strong Motion Instrumentation Program</td>
</tr>
<tr>
<td>d</td>
<td>Bolt Diameter</td>
</tr>
<tr>
<td>DAF</td>
<td>Dynamic Amplification Factor</td>
</tr>
<tr>
<td>DCDT</td>
<td>Direct Current Differential Transformer</td>
</tr>
<tr>
<td>D_s</td>
<td>Width of the Braced Frame</td>
</tr>
<tr>
<td>Δ_u</td>
<td>Ultimate Deformation</td>
</tr>
<tr>
<td>Δ_y</td>
<td>Yield Deformation</td>
</tr>
<tr>
<td>e</td>
<td>End Distance</td>
</tr>
<tr>
<td>F</td>
<td>Foundation Factor</td>
</tr>
<tr>
<td>FAS</td>
<td>Fourier Amplitude Spectrum</td>
</tr>
<tr>
<td>F_d</td>
<td>Design Force in the Brace</td>
</tr>
<tr>
<td>F_max</td>
<td>Maximum Force (Capacity) of the Connection</td>
</tr>
<tr>
<td>FRF</td>
<td>Frequency Response Function</td>
</tr>
<tr>
<td>F_y</td>
<td>Connection Yield Force</td>
</tr>
<tr>
<td>g</td>
<td>Acceleration due to Gravity</td>
</tr>
<tr>
<td>H</td>
<td>Total Height of the Braced Frame</td>
</tr>
<tr>
<td>HSS</td>
<td>Hollow Structural Section</td>
</tr>
<tr>
<td>Notation</td>
<td>Description</td>
</tr>
<tr>
<td>----------</td>
<td>--------------------------------------------------</td>
</tr>
<tr>
<td>I</td>
<td>Importance Factor</td>
</tr>
<tr>
<td>K&lt;sub&gt;i&lt;/sub&gt;</td>
<td>Initial Connection Stiffness</td>
</tr>
<tr>
<td>K&lt;sub&gt;d&lt;/sub&gt;</td>
<td>Load Duration Factor</td>
</tr>
<tr>
<td>L</td>
<td>Brace Length</td>
</tr>
<tr>
<td>LVDT</td>
<td>Linear Variable Displacement Transformer</td>
</tr>
<tr>
<td>LVL</td>
<td>Laminated Veneer Lumber</td>
</tr>
<tr>
<td>μ</td>
<td>Displacement Ductility</td>
</tr>
<tr>
<td>m</td>
<td>Mass</td>
</tr>
<tr>
<td>M</td>
<td>Magnitude</td>
</tr>
<tr>
<td>MDOF</td>
<td>Multi-Degree-of-Freedom</td>
</tr>
<tr>
<td>MOE</td>
<td>Modulus of Elasticity</td>
</tr>
<tr>
<td>NBCC</td>
<td>National Building Code of Canada</td>
</tr>
<tr>
<td>OSB</td>
<td>Oriented Strand Board</td>
</tr>
<tr>
<td>P&lt;sub&gt;0&lt;/sub&gt;</td>
<td>Force at Zero Deformation in a Connection</td>
</tr>
<tr>
<td>PGA</td>
<td>Peak Ground Acceleration</td>
</tr>
<tr>
<td>PGV</td>
<td>Peak Ground Velocity</td>
</tr>
<tr>
<td>PSL</td>
<td>Parallel Strand Lumber</td>
</tr>
<tr>
<td>R</td>
<td>Force Modification Factor</td>
</tr>
<tr>
<td>S</td>
<td>Seismic Response Factor</td>
</tr>
<tr>
<td>s</td>
<td>Spacing Between Fasteners</td>
</tr>
<tr>
<td>SDOF</td>
<td>Single Degree of Freedom</td>
</tr>
<tr>
<td>SPF</td>
<td>Spruce-Pine-Fir</td>
</tr>
<tr>
<td>S&lt;sub&gt;r&lt;/sub&gt;</td>
<td>Row Spacing</td>
</tr>
<tr>
<td>T</td>
<td>Fundamental Period of a Structure</td>
</tr>
<tr>
<td>U</td>
<td>Calibration Factor</td>
</tr>
<tr>
<td>V</td>
<td>Total Design Lateral Load</td>
</tr>
<tr>
<td>v</td>
<td>Zonal Velocity Ratio</td>
</tr>
<tr>
<td>V&lt;sub&gt;e&lt;/sub&gt;</td>
<td>Seismic Force Representing Elastic Response</td>
</tr>
<tr>
<td>W</td>
<td>Estimated Seismic Weight of the Building</td>
</tr>
</tbody>
</table>
ACKNOWLEDGEMENTS

The project was conducted with financial and in-kind support from the Science Council of British Columbia, the Department of Civil Engineering at the University of British Columbia, Forest Renewal British Columbia and Forintek Canada Corp. The support of all institutions is gratefully acknowledged.

There have been so many people who have assisted me during the various tasks associated with the completion of this dissertation, that I cannot hope to mention all of them here. I hope that no one will be offended if I neglect to acknowledge their help individually. It was very much appreciated, and I would not have been able to accomplish as much without their assistance.

I would like first to thank my academic supervisor Dr. Helmut G. L. Prion for the thoughtful guidance, technical advice and financial support he has given me over the years. I am grateful for the personal advice, encouragement and understanding he has shown during some difficult times. I also want to thank him for giving me the opportunity to be involved in the education process in the Department. In addition, his valuable comments and suggestions during the thorough review of the manuscript are sincerely appreciated.

Professional advice, guidance and comments in various stages of the project from Mr. Erol Karacabeyli, senior scientist from Forintek Canada Corp, is greatly acknowledged. I also thank him for providing an additional financial support for the project and excellent working conditions.

I would like to thank Dr. Ricardo Foschi, Dr. Carlos Ventura, Dr. Frank Lam, Dr. Don Anderson and Dr. Robert Sexsmith from UBC, for their professional advice during the research
project. Dr. Foschi and Dr. Ventura are especially thanked for their personal encouragement and support during my tenure at UBC.

The experimental section of this thesis would not have been possible without the assistance of the laboratory technologists and technicians. Special thanks are extended to Link Olson and Rollie Wakeman, senior technologists from Forintek, for their deep involvement in the research program. I am especially grateful to Link Olson for his help and advice on the experimental setup for the quasi-static and shake table tests. Howard Nicol from UBC is thanked for his time and valuable hints during the shaking table tests. I also thank Doug Hudniuk for his machining and welding help with the experimental models.

Several undergraduate and graduate students contributed to the successful contribution of the project. Among all these, I would like to thank former graduate students Vincent Latendresse and Jachym Rudolf for their hints about the signal processing procedures.

I would like to express my deepest gratitude to my wife Vesna and my son Daniel. Vesna is given special thanks not only for her emotional support during my entire graduate career, but also for the patience, love and understanding she has shown during the past six years. I am especially proud of my son Daniel for his adult-like understanding of the reasons why his daddy had to be away from him for so many long months.

Finally, but certainly not least, I would like to take this opportunity to thank my parents, Aleksandra and Lazar, who taught me that accomplishing anything worthwhile requires determination and perseverance.
To Daniel and Vesna
STRUCTURAL ENGINEERING IS

THE ART OF USING MATERIALS

that have properties which can only be estimated

TO BUILD REAL STRUCTURES

that can only be approximately analyzed

TO WITHSTAND FORCES

that are not accurately known

SO THAT OUR RESPONSIBILITY WITH RESPECT TO

PUBLIC SAFETY IS SATISFIED

Adapted From an Unknown Author
1. INTRODUCTION

1.1 PROBLEM OVERVIEW

Among the natural hazards faced by humans on earth, an earthquake is probably one the most disastrous and unpredictable actions. Many of the heavily populated regions of the world are in seismically active regions. The occurrence of an earthquake there often results not only in enormous material damage but also in substantial loss of life and in physical and psychic trauma. On average, about 10,000 people die every year because of earthquakes around the world, with property damage of approximately $1 billion (Naeim, 1989). Because earthquakes as natural phenomena are unpreventable and unforeseen, the only course of action for engineers to take is to build seismically resistant structures. These structures should be designed to behave in desirable and predictable manner when subjected to earthquake ground motion.

The first step required in a design of earthquake resistant structures is acquiring a thorough understanding of the behaviour of a structure during an earthquake. Experiences from areas that experienced severe earthquakes indicate that timber structures exhibit good seismic performance when carefully designed. This is due to a number of factors, particularly the high strength-to-weight ratio of timber as a material and due to its enhanced strength under short term loading. Structural failures have been observed, however, due to poor materials or design or overlooking
Introduction

some simple but fundamental principles. Despite the qualities and advantages, timber has been
the material least used for engineering purposes when compared to other structural materials,
even in regions where timber is a plentiful resource. In addition, very little attention is given to
timber structures at international conferences on structures or in national building codes. This
apparent reluctance of using timber structures in some of the seismic prone regions in the world
is often influenced by the lack of adequate research and other difficulties in support to the
designing and fabricating process of ductile structural connections. Reservations regarding the
structural resistance to an event of a fire, as one of the post earthquake phenomena, may have a
contribution as well.

A braced timber frame is one of the most efficient structural systems to resist lateral forces
induced by strong earthquakes or winds. The system is used in conventional and glued-
laminated timber construction for industrial, multipurpose or residential use. Bracing essentially
provides triangulation by means of diagonal members inserted in the rectangular bays of the
frame. Diagonal braces, however, have the disadvantage of obstructing movement of people and
goods and are usually located in the plane of the wall. In concentrically braced frames, there is
essentially no eccentricity in the joints and the lateral forces are resisted by almost pure axial
loads in the braces.

The seismic response of a braced timber structure in general is a complex issue, involving many
different interacting factors, which need to be understood and quantified. One of the most
important considerations is to provide a system that can absorb large amounts of energy and
thus lower the earthquake-induced forces, while maintaining adequate stiffness to avoid
excessive deformations. A significant deflection of the frame is only possible when there are
considerable deformations in the joints. To satisfy all these requirements, the seismic design process must include connections that have a careful balance of strength, stiffness and ductility. Ductile connections able to sustain large non-linear deformations are preferred because they limit the force level in the braces.

A limited amount of information is currently available on the seismic behaviour of braced timber frames. In general, the static and cyclic behaviour of bolted connections has been studied and relatively good performance was obtained in connections with small diameter bolts. Some of the results on these connections can potentially be used for braced timber frames although the performance of bolted connections under dynamic loading has hardly been fully understood. The connection influence on the dynamic behaviour of the braced frames has been a concern to only a handful of researchers around the world. In addition, the dynamic behaviour of frames with different types of connectors such as glulam rivets or dowels has not been explored. As a result the seismic design process of braced timber frames, is based more on experience from previous structural performance than on hard scientific evidence.

Force modification factors in The National Building Code of Canada (NBCC) and other foreign national design codes, seem to be based on general observation only and do not address the behaviour of the frame components as such, which could behave either in a brittle or a ductile manner. Similar to the provisions in the design codes, guidelines are required that specify the framing details for ductile braced frames. The lack of design information demonstrates a need for further experimental testing to provide information that can be used directly to improve the existing seismic design procedures. Mathematical models for the seismic analysis of braced timber frames are also needed to extend the application of experimental results, predict the
structural behaviour under specified seismic loading and perform parametric studies on their seismic performance.

1.2 RESEARCH OBJECTIVES

For the reasons mentioned above, a research project on seismic performance of braced timber frames was initiated in Canada several years ago. The study was conducted at The Department of Civil Engineering at UBC in collaboration with Forintek Canada Corp. staff. The results of the study on braced timber frames which are presented in this thesis, will also be incorporated in a seismic research program entitled “Lateral Resistance of Engineered Wood Structures to Seismic and Wind Loads” by Forintek Canada Corp. The main objectives of the project on braced timber frames can be summarized as:

- Increase the body of knowledge on the dynamic performance of braced timber frames;
- Determine the factors that influence the seismic behaviour of braced timber frames;
- Study the influence of different connection details on seismic behaviour of these frames;
- Develop an analytical procedure to obtain corresponding force modification factors for braced timber frames with different connections;
- Provide designers and code officials with relevant information about design details needed to improve the seismic behaviour of braced timber frames.

1.3 SCOPE

In order to meet the research objectives stated above, the research program was planned to consist of five parts:
Introduction

1. Experimental study of the cyclic behaviour of different connections;

2. Shake table tests on simplified single storey braced frame models;

3. Analytical study to verify computer models for dynamic behaviour;

4. Shake table testing of two-storey braced frame model;

5. Analytical study to expand the tests results to different design conditions and establish the appropriate force reduction factors.

Because the seismic behaviour of braced timber frames is mostly governed by the buckling properties of the braces and the failure (ductility) properties of the brace end connections, an emphasis in the experimental part was placed on the behaviour of different types of brace connections. Using glued-laminated timber (glulam) as a material for the frame, monotonic static and cyclic tests were conducted on a variety of connections with different diameter bolts or high strength glulam rivets with steel side plates. The hysteresis curves obtained during the cyclic tests were then used to develop non-linear mathematical models for all the connections tested.

Besides cyclic connection tests, shaking table tests were subsequently conducted on a selected number of single story braced frame models. Results from these tests were used to verify the existing analytical models for different connections and compare the connection hysteretic response with the one obtained during quasi-static tests. The verified models were later incorporated in the DRAIN 2DX computer program for non-linear static and dynamic analysis of structures. Structural analytical models were also defined and numerous non-linear static and time history dynamic analyses were performed. From these analyses, it was possible to determine the influence of different connection details on the seismic response of the frames.
Based on the analyses, an estimate was made of the appropriate force modification factors for earthquake resistant design of braced timber frames in NBCC.

To examine the dynamic behaviour of a multi-storey braced frame structure, static-push-over and shaking table tests on a two-storey braced frame model were conducted. The wood elements used in the model were glued laminated members with the same cross section and properties as those used throughout the research program before. Glulam rivets were used as fasteners for all the connections. Three different earthquake records were used during the tests, scaled to various acceleration levels.

1.4 THESIS OUTLINE

The thesis presents the different steps followed in the study to achieve the research objectives. A background on the wood as a structural material, performance of timber structures during recent earthquakes and the corresponding literature review on problems concerning braced timber frames are given in the second chapter. Chapter three describes the quasi-static tests conducted on various connections used in braced timber frames. Results and discussions on test results include load-deformation characteristics, maximum loads, ductility levels and energy dissipating characteristics. In chapter four the objectives and experimental program details for the single brace shake table tests are presented. Experimental results presented here include time history parameters of the dynamic response, analysis of results in the frequency domain and comparison of cyclic and shake table tests.
Analytical predictions of the single brace shake table tests including the introduction of linear and non-linear mathematical models is given in chapter five. Results from non-linear static and dynamic analyses of braced frames with different connections are given in chapter six. Results include the deformation demands for frames with different connections, force modification factors and parameters that influence them. Chapter seven outlines the objectives, testing sequence, and results from shaking table tests on a two-storey braced frame model. Test results include time history parameters of the dynamic response, failure mechanism and damage progress assessment as well as frequency content of the response. Analytical predictions of the shake table tests on the two-storey model are presented in chapter eight. Finally, in chapter nine a summary on the results of the study is given. The chapter also provides recommendations for further research. A list of references is given in chapter ten in the thesis.
2. LITERATURE REVIEW

2.1 BRIEF HISTORY OF TIMBER STRUCTURES

Timber has been available as a constructional material to most societies since the human race started to build crude shelters at the beginning of civilization. It was used in the construction of buildings, bridges, machinery, civil engineering works, boats etc. since mankind first learned to fashion tools. The use of wood throughout history was extensive, mainly due to its availability, workability, lightness and ease of construction.

Probably the earliest "wood structures" constructed by mankind were the shelters made from a framework of tree branches covered with leaves or animal hides. In the densely forested regions of Europe, different house building techniques were developed that date back to the Stone Age. One of the most revolutionary techniques developed in the past in Europe was the log notching technique. According to this method, structural stability was provided by notching the round logs at the corner intersections, so that the wall planes of the house interlock. The earliest evidence of notched log-houses in Europe dates back to around 800 BC and was not restricted to small buildings. This technique was used in Europe up to the 11th century, for example in the churches at Greensted in England and Lund in Denmark (Chilton 1995). The log cabin was later widely used in the forested areas of USA and Canada where rapid colonisation required
construction of a large quantity of residential accommodation using relatively unskilled labour in a short time.

The mysterious and outstanding Norwegian stave churches (1150-1350) built at the same time as the European cathedrals and the loghouses in northern Europe, are an interesting chapter of the history of timber structures (Aass, 1996). Up to the 14th century, it is estimated that between 800 and 1000 stave churches were built in Norway. The name derives from the substantial timber columns or staves, which are a dominant structural element in their construction. Twenty-nine churches still stand today, some 800 years later, in a tribute to the builder’s skill in the selection of durable materials, a stable structural system and construction details appropriate for the wet and cold northern climate.

![Diagram of a typical Chinese building system of 13th century.](image)

Figure 2.1. Typical Chinese building system of 13th century.

A very sophisticated modular timber building system was developed and documented over 1000 years ago in China. The documentation of the system called “Ying-tsao Fa-shih” was a detailed manual of building procedure and practice that, although revised by subsequent rulers, survived in modified form until the founding of the Chinese Republic in 1912 (Needham, 1971). The
basic structure of a traditional Chinese building consisted of a grid of timber columns founded on large stone bases, which then supported the floor beams and the heavy roof construction (Figure 2.1).

In Japan, there is also a long tradition of using timber structures and many historic buildings were constructed following the example of the Chinese building system. One of these buildings is the largest ancient timber structure in the world, the Todaiji temple at Nara, which is 57 m wide by 50 m deep and 47 m high. The quality of traditional timber joinery in Japan was high and many complex and inventive joints were used to connect structural elements without using metal fasteners. Because of their complex joinery, the Japanese temples were able to sustain large deformations and dissipate a part of the seismic input energy, which helped them survive many severe earthquakes.

![Diagram of box frame timber system](image)

Figure 2.2. Box frame timber system used in medieval Europe; a) Tie beam, b) wall plate, c) wall post, d) bressummer, e) storey plate, f) sole plate, g) summer, h) girder.

In Europe during medieval times, timber-framed construction was commonly used for houses, churches, schools, barns and many other structures. The box framing system, for example,
(Figure 2.2) was basically post and beam construction with an addition of diagonal bracing within the wall planes to resist lateral forces. Generally, all the joints were made using hardwood dowels. A common feature of this construction method was that the upper-storey cantilevered a short distance out from the walls of the ground floor. In densely populated medieval cities of Europe this provided additional accommodation over the narrow streets, and improved the structural efficiency of the beams.

During the Renaissance and Post-Renaissance period, a significant step forward in building and design of timber structures was recorded. A number of books were published with examples of new thinking and state-of-the-art writing. During this period, triangulated wooden trusses for large spans were recommended and the beautiful Swiss Renaissance bridges were built, many still existing and in use for heavy traffic. Many Japanese temples and pagodas were built, as well as the famous wooden suspension bridges in China and Peru. Massive timbers were used in roof construction in the great cathedrals of Europe. In the USA the first modern timber bridges were built. Early American bridges usually consisted of a simple beam and pile system like the Great Bridge over the Charles River, (1660), and the 83 m York River Bridge in Maine built in 1761 (Chilton 1995).

By the 19th century, laminated timber structures were in common use in many European countries. One of the first major uses in a building was the train-shed roof of Kings Cross Station in London, England. The roof was constructed using laminated timber arches of 30.5 m span, with a 600 mm depth of the cross section. Another example is the beautiful “Colossus Bridge” over the Schuylkill River in Philadelphia, built in 1812. This was a fully covered, trussed-arched, timber structure of 103.7 m clear span, with laminated arch chords of 330 mm
by 1030 mm in cross section (ASCE, 1976). According to many critics, the architectural beauty of the bridge and its bold and scientific design probably was never surpassed in America. Early laminated timber structures used mechanical methods, bolts, dowels, etc. to connect the individual laminations. The development of synthetic resin glues, however, enabled production of the first glued laminated members in 1906.

The twentieth century brought many innovations, especially in the area of timber connections. This extended the range of applications and today many long span structures pay tribute to timber as a constructional material. Connections such as the Bulldog® (Norway, 1920) or Greim-type multi-layer connection from Germany (1930), are still used today in their original or modified versions in famous structures such as the Lillehammer Olympic arena and the new airport building at Gardemoen in Norway. The punched metal plate connector patented in USA in 1950s revolutionized the use of prefabricated light timber trusses.

In 1942, several airship hangars were constructed for the US Navy. These trussed-arch hangars (305 m long, 91 m wide and 52 m high) are still some of the largest clear span timber structures in the world. Since the 1950s, timber has also been used in the construction of shell or dome structures that fully utilize their 3-D form to resist applied loads. Timber is particularly well suited for construction of shells based on hyperbolic paraboloid geometry as this double curved surface can be generated using straight elements.

In North America today, wood is still by far the primary construction material for residential housing, used as sawn lumber in framed construction. The most common timber-framed construction method is the so-called light frame platform construction where storey-height wall
panels are set on the intermediate floor platforms. Timber is also used in construction of commercial and industrial structures, although this application is not as frequent. The timber construction industry today is facing several challenges mainly because of projected shortage and thus price increase of high quality lumber and vigorous campaigning of the steel industry. In the residential area, the principal challenge is the preparation of rational design standards for structural systems and the technical support required to implement them. Commercial and industrial markets present even more challenges, such as design methodology, material research, computer design aids and an education process in support of the design.

2.2 TIMBER AS A STRUCTURAL MATERIAL

Wood is an abundant and user-friendly material, different from most other engineering materials in the sense that the user has little influence on its material properties. When used properly, it is a cost efficient material for a variety of structures such as large span roof structures, bridges, residential, office or industrial buildings. In an environmentally conscious society, building with wood has become an important issue. Since wood contains far less embodied energy than most other materials such as steel, aluminum or concrete, greater use of wood means less construction energy needed, which becomes a very important economical and environmental issue (Buchanan, 1994).

Timber is light in weight, thus requiring less heavy load machinery during construction. This characteristic, combined with the softness of the material, allows for implementation of simple construction techniques and the possibility of construction with fewer workers. Contrary to common belief, the fire resistance of many wood structures is often considered an advantage of
this material. For large timber sections, the burning mechanism consists of charring around the
cross-section, which insulates the member and slows the rate of material deterioration due to
burning. This mechanism helps the members keep their structural integrity and capacity, unlike
other materials such as steel, for example, which experiences a loss of stiffness at temperatures
as low as 300° C.

Wood is an orthotropic material, which means that the material properties can be defined in the
three mutually perpendicular directions: longitudinal, transverse and radial. The strength and
stiffness properties of wood vary with the relative orientation of the applied load to the wood
fibers, which cause the mechanisms of failure to vary accordingly. Figure 2.3 shows typical
stress-strain relationships for timber in tension and compression, both parallel and perpendicular
to grain.

![Stress-strain relationships for wood](Buchanan, 1994)

Figure 2.3. Stress-strain relationships for wood (Buchanan, 1994).
As presented in the figure, tension failures either parallel or perpendicular to grain tend to be abrupt, so these must be avoided, particularly under seismic loading. All the failures in shear and some of the failures in bending are also brittle and should also be avoided. The type of bending failure depends on the ratio of compression to tension strength, while column behaviour can be brittle or ductile depending on the ratio of compression to bending stress. Only the compression parallel and perpendicular to grain failures show a ductile behaviour.

The duration of loading has an important effect on the ultimate strength of wood. The ultimate strength of wood decreases under long duration load. In addition, the moisture content can affect the properties and strength of wood members. An increased moisture content in wood reduces the modulus of elasticity, modulus of rupture and compression strength parallel to grain. Only toughness of wood increases with an increase in moisture because it involves ductility and strength, and dry wood is less ductile than green wood.

Finally, variability of material properties is an important intrinsic characteristic of wood as a material. In addition to knots and other defects, factors such as the rate of growth, growing conditions and species affect the material properties. The proper grading of wood is therefore of utmost importance to select the material according to its end use. Extensive research has been undertaken to optimize the properties of wood by transforming it into various engineering wood products, and developing new design techniques (Williams, 1999). Plywood and oriented strand board (OSB), which are used as panels in structural walls, were the first products on the market. Another widely used engineered wood product is glued laminated timber (glulam). Glulam is manufactured by finger jointing of dimension lumber, which is then coated with adhesive and glued together to achieve a larger laminated section. Because the wood defects are not aligned
for all the lamina, the glulam cross section has less strength variability compared to the cross section of sawn lumber of the same size.

More recently other products such as laminated veneer lumber (LVL), laminated strand lumber (LSL) and parallel strand lumber (PSL) were developed. Laminated veneer lumber is composed of several layers of wood veneer placed with the same grain orientation, which are then glued together under pressure and heat. Because of smaller cross sections, LVL is mostly used for beam elements. LSL elements are produced of long flaked strands (up to 300 mm) which are then coated with resin and pressed into large billets by a process that includes steam injection. PSL elements are composed of small wood strands (up to 15 mm long) oriented in the longitudinal direction. The strands are coated with a waterproof adhesive, pressed together and cured using a microwave process to form a rectangular section. The manufacturer of these products in North America is Trus Joist MacMillan and the registered names are Microllam® for LVL and Parallam® for PSL. Their higher design strengths and the range of available dimensions make them desirable for a large range of residential and commercial construction. Some examples where Parallam® was used in non-residential applications in BC include the South Surrey Ice Arena (1990), the University of Northern BC (1994), the Seabird Island School (1992), the FERIC Building (1991) and the Forintek Canada Corp. Western Laboratory in Vancouver (1990) and the Forest Sciences Building at the University of BC (1998).

2.3 PAST SEISMIC PERFORMANCE OF TIMBER BUILDINGS

Many timber buildings that were constructed without the benefit of proper engineering design or before engineering calculations were established, have generally performed well during past
earthquakes. Old residential timber buildings in the Mediterranean region and pagodas and temples in Japan and China have survived 1,000 or more years of use and environmental loading including strong earthquakes (Foliente, 1996).

Most Chinese pagodas and temples performed well because of their symmetrical plan (many in two axes), almost uniform elevation, their light weight (attract lower inertial force), integrity of beams, columns and roof (with effective joints, braces and ties), redundancy (some pagodas are supported by 32 columns) and use of effective joining systems called brackets. The latter have been used extensively in Chinese and Japanese temple structures. There is no relative movement between elements in the bracket system when there is a small horizontal force acting on it. When the lateral load is big enough, it forces the elements to slide along each other, dissipating energy in the process, much like the modern sliding friction devices (Hu, 1991). The Japanese pagodas have the same features as above, and those built during the 17th century had, in addition, a suspended central column which was independent of the surrounding structural frames (Tahabashi, 1960). Under seismic ground motion, the column acts as a suspended pendulum, which is a passive control device that reduces the structural response.

Single family dwellings using the traditional North American light-frame (platform) construction system have performed relatively well during the strong earthquakes of this century. Some of this performance can be attributed to the material characteristics of wood itself, and some is a result of the lightness and high redundancy of the system. The idea, however, that “if the structure is made of timber, it is safe against an earthquake”, is a myth. While wood framed buildings performed “generally well” in the past, there have always been wood buildings that performed badly, especially larger buildings and those with irregular
shapes. By inspecting those buildings after a major earthquake, we can learn from the failures and introduce changes to the design and construction procedure in the future.

Observations from the 1964 Alaskan Earthquake had shown that some failures resulted from the lack of structural integrity. Corner connections, floor to wall as well as wall to roof connections, were found inadequate of maintaining a structure to act as a unit. In many cases, inadequate lateral resistance was provided (Soltis et al. 1980). Experiences from the 1971 San Fernando Earthquake in California indicated that buildings with inadequate lateral bracing such as two storey residences with large garage openings on the lower floor, suffered extensive damage. The greatest deficiency of the wood frame construction was observed to be its lack of resistance to torsional racking caused by large openings and unsymmetrical building plans. This was typically case in bottom storey, leading to the so called soft first storey effect. Following the San Fernando Earthquake, significant changes were made to the Uniform Building Code (UBC) which is used throughout the western USA.

The 1983 Coalinga, California Earthquake pointed out some other characteristic failures of timber structures. Extensive roof damage was found in a majority of buildings due to lack of roof diaphragms and because of bad connections between roof segments and between the roof and the wall. Some houses which had masonry bearing walls suffered extensive damage to the masonry that resulted in additional stresses to the rest of timber structure (Rihal, 1984). The assessments from other earthquakes in California such as the Whittier Narrows Earthquake 1987, Loma Prieta, 1989 (O'Halloran, 1990), and Northridge in 1994 (Stieda, 1994), showed that without adequate anchorage the houses slid off their foundations. In addition, the lack of lateral support of porch columns often resulted in a partial roof collapse. During the Northridge
earthquake more than 50% of the total damage was incurred in wood frame construction. This has led to a major investigation into the performance of wood frame construction in high risk earthquake zones.

Valuable lessons from past seismic performance of timber structures can be learned from the recent Japanese earthquakes. The 1974 Off Miyagi-ken and 1978 Off Izu peninsula earthquakes pointed out the soft storey problem once again, just years after San Fernando (Iizuka, 1980). Furthermore, extensive damage was done to traditional Japanese post-and-beam houses during the 1995 Hyogo-ken Nanby earthquake (Prion et al. 1995). Heavy roofs, inadequate lateral bracing, lack of shear wall panels, material decay and soft stories were the primary cause of collapse of nearly 150,000 wooden buildings, killing and injuring thousands of people and leaving over 300,000 people homeless.

Among many reports on earthquake damage, however, it is very difficult to obtain any information on failures of larger timber structures with braced frames or moment resisting frames as a lateral load resisting system. This is mainly because these construction systems were not used often in the past, and even if they were, it was mostly in areas not prone to severe earthquakes.

### 2.4 SEISMIC BEHAVIOUR OF TIMBER STRUCTURES

The response of timber structures to earthquake ground motion depends on many different properties of the components that make up the system. The main components that need to be considered are the material itself and the fasteners connecting the timber members. As shown
before (Figure 2.3), timber members in axial tension or in bending exhibit a brittle behaviour, while those in compression perpendicular or parallel to grain, experience a ductile behaviour. The effect of connections on the structural response depends on the nature of the fasteners used and whether the timber or the fasteners governs the behaviour. For example, nails, dowels or bolts in the connections can be designed to remain linear elastic or yield during the seismic action, so the corresponding structures can have a linear or a non-linear ductile response. The influence of different connections used in timber structures in seismic areas, as well as their properties, will be discussed in the next section.

The amount of damping in a structure significantly affects its response to dynamic loading. Essentially four sources of damping are evident in timber structures: (i) Damping within the micro structure of the wood itself; (ii) Slip damping at surfaces in contact at joints and connections; (iii) Damping provided by special adhesive layers in glued joints and (iv) Hysteretic damping of connections (Yeh et al. 1971). Several studies reported the total equivalent damping, expressed in terms of equivalent viscous damping, to vary from 3% to 20% of the critical damping, depending on the structural system. For example, Medearis found that the equivalent viscous damping in sheathed diaphragms to be 8% - 10% regardless of amplitude (Dowrick, 1992). These damping values show that a large amount of energy can be absorbed during an earthquake.

The static load-displacement curves of ductile timber structures are smooth and continuous from the onset of loading to the ultimate load. Compared to the curves of other materials (steel, for example), there is no clearly defined yielding point, thus a much more flexible definition of yielding and ductility is used. The hysteretic load-deformation curves of timber joints or
components are mildly to severely pinched, depending on the fasteners used (Figure 2.4). The
pinching effect results in thinner loops in the middle compared to near the ends. This
phenomenon is caused by the loss of stiffness at small joint slips, where a cavity around the
fastener is formed by crushing of the wood. The fastener shank, without wood support within
the deformation level of the slip, provides the sole resistance to the applied load. The stiffness of
the connection again increases gradually as the fastener makes contacts with the surrounding
wood at higher deformations.

Figure 2.4. Typical hysteretic behavior of a timber structure component.

Considering that the area inside the hysteresis loop for each cycle represents the amount of
energy dissipated during that cycle, the pinching effect reduces the hysteretic damping of the
structure. Some recent test results have shown, however, that beside the shape of the hysteresis
curve (pinching), the ability of timber structures to sustain large deformations without
significant strength deterioration is also very significant (Buchanan et al. 1988). Deam and King
(1994) supported this statement after considering the low damage ratio experienced by timber
buildings in past earthquakes.
A progressive reduction of stiffness in each loading cycle (stiffness degradation) and reduction of strength when cyclically loaded to the same displacement level (strength degradation) are very common characteristics of the load-deformation (hysteretic) response of timber structures under repetitive cyclic loads. A very important but still controversial topic on the response of timber structures is that the response at a given time depends not only on instantaneous displacements, but also on past history or the input and response at some earlier time. This phenomenon also is known as memory, was observed in nailed and bolted timber joints under irregular short or medium term lateral loading have memory (Whale 1988; Foliente, 1995).

The relatively low stiffness of wood and timber structures in general is of benefit for their seismic response. This leads to structures with longer natural periods of vibration thus avoiding resonance in typical frequency ranges of most earthquakes. Nevertheless, most timber structures are relatively short, only one or two stories in height, and lateral deflection criteria often govern the design requiring a stiff structure, so the benefits of having a more flexible structure are usually lost.

Finally, the response of timber structures was found to be profoundly affected by foundation effects. Timber buildings appeared to suffer more earthquake damage when located on soft soil than on bedrock. This contradicts the fact that most timber buildings have an initial fundamental period of vibration in the range of 0.1 to 0.8 sec, which would indicate that resonance with the soil would be more likely on a firmer rather than on a softer soil. Nevertheless, timber buildings tend to lose some of the stiffness at the joints, resulting in an increase of their natural periods over time, especially if some previous shaking had occurred. With the softer structures,
resonance is therefore more probable on a thicker and softer layer of soil. Consequently, for timber structures built on soft soil, extra measures have to be taken to ensure structural integrity, particularly at the foundation level.

2.5 CONNECTORS FOR TIMBER STRUCTURES IN SEISMIC REGIONS

2.5.1 Important Seismic Considerations

The main challenge in the seismic design of timber structures is the design and construction of appropriate connections. Because of the cyclic nature of the loading, the wood will inevitably be stressed in a direction of weak strength, which presents a particularly challenging problem. The fact that most collapses of timber structures during earthquakes were associated with connection failures emphasizes the importance of studying the behaviour of different types of connections under cyclic loading. To obtain ductile structural behaviour, it is therefore of great importance to use ductile connections that are weaker than the wood members they connect. The hysteretic behaviour of connections is governed by the material properties of the fastener and by the embedding behaviour of wood. Mild steel connectors with a large deformation capacity are generally more suitable for energy dissipation than high strength steel connectors with a brittle failure mode.

As mentioned earlier, the properties of wood vary with the orientation of loading and therefore the failure mechanism is more complex than for isotropic materials. Consequently, connections that stress the wood in tension perpendicular to the grain have to be avoided, because they can result in brittle failures, even at very low loads. The ability to recognize potential failures of this type in connections is one of the most appreciated skills of earthquake resistant timber design.
Several different factors influence the behaviour of connections in timber structures. Tests showed that the moisture content of wood has a significant influence on the strength of dowel type connectors (Welchert and Hinkle, 1966). Consequently, in the Canadian Code for Engineering Design in Wood (CSA-O86.1-M94), the capacity of timber connections under wet conditions is reduced to allow for the loss of strength of wood and for the probability of splitting caused by shrinkage due to moisture content variations.

The group effect is also an important factor to be considered in timber connections. For a small group of fasteners, the total strength is approximately equal to the product of the strength of a single fastener by the number of fasteners. For a larger group of fasteners the capacity of the connection is reduced due to a group effect (Lantos, 1969). The relative rigidity of the wood compared to the fasteners is the main factor to influence the group effect. Under linear elastic conditions, the load distribution in a group of fasteners is very uneven. When yielding of connectors occurs, it allows for redistribution of the load throughout all the fasteners, resulting in a more equal load sharing and reduced group effect.

The influence of the rate of loading on the strength and behaviour of timber connections was studied by a number of researchers. McLain (1975) tested single-nail joints with a variety of member combinations and found no statistically significant difference in load-deformation relationships between static and low frequency cyclic loading. Nailed and bolted wood-to-wood joints, on the other hand, were found to be considerably stiffer under rapid loading than under static loading (Wilkinson, 1976). His results also showed higher resistance at the same displacement level under dynamic loading, which indicates an apparent rate of loading effect.
Girhammar and Anderson (1988) tested nailed joints monotonically considering two wood species and five deformation rates (from 2 mm/min to 1250 mm/min). They observed that the stiffness in the elastic range was not influenced by the deformation rate. The results also showed that the rate of loading effect was greater for wood bearing than for nail bending. Bodig and Farquhar (1988) observed from their tests that the load duration mostly affected nailed joints, followed by metal plate connectors and bolted connections.

Fatigue, which is a tendency of a material to break under repeated stress, could be a problem in timber connections as it reduces the short term or residual connection strength due to cumulative damage after a period of cyclic loading. Fatigue was observed to be a potential problem in joints with high-density members or members with steel side plates (Williams, 1984; Dolan and Madsen 1992; Ni and Chui 1994). So far it is known only that the fatigue of a connector is very much dependent on the stress levels induced on the connection and the number of load cycles. Fatigue problems in some joint or assembly configurations under earthquake-type loading need additional research. Critical joint configurations and material combinations that may be susceptible to fatigue need to be identified.

2.5.2 Connector Types and Performance Investigation

Nowadays, the most common connectors can be divided into three main categories, namely dowel type, surface type and bearing type. Dowel type connectors are cylindrically shaped connectors that penetrate deep into the wood and transfer the load between members by a combination of wood bearing and connector bending. This type of connector includes nails, spikes, staples, bolts, pins and screws. Surface type connectors combine the dowel-type action
with the metal plates so they can collect and transfer the load near the surface of the member. Surface type connectors include punched metal plates, tooth plates, glulam rivets and Bulldog® connectors. Bearing type fasteners are designed to transfer the forces relying solely on the shear or bearing resistance of the wood, parallel or perpendicular to grain. Examples include split rings, tooth rings, and shear plates. Extensive experimental research has been conducted to determine the performance of different connections, although almost exclusively under static monotonic loading. The interest in the cyclic properties of building components is fairly recent.

2.5.2.1 Nails

Nails loaded in shear are typically ductile connectors capable of resisting many reversals of cyclic loading. Under cyclic loading they usually develop slackness and pinched hysteresis loops as the nails bend and cause local crushing of the adjacent wood fibers (Figure 2.5.a). Large displacements, however, can be achieved without connection failure, even though some pullout of the nail is to be expected (Figure 2.5.b). Buchanan and Dean (1988) recommended a minimum nail length to avoid a premature withdrawal. Several parameters such as wood density, nail length, nail strength, spacing etc. influence the connection behaviour. Static tests on nailed connections showed that increased nailing density (smaller spacing) in the connection leads to formation of a brittle block-tearing failure mode with reduced capacity (Kangas, 1999). Cyclic tests also showed that nailed connections containing high density wood members may experience failure due to fatigue breakage of the nails (Ni et al. 1994).
Some of the first researchers to study the damping and stiffness properties of nailed joints under cyclic loading were Kaneta (1958) and Jacobsen (1960). They observed that the specimen's load-deformation curve from cyclic tests falls below the curve obtained from the monotonic static test (Foliente, 1994). Polansek and Bastendorff (1987) tested twenty-one joint types with fifteen specimens each and found out that, among construction variables, lumber species affected the equivalent viscous damping and stiffness the most.

Moment resisting connections using nails were tested by several researchers (Buchanan et al. 1988; Ceccotti et al. 1994). They found the hysteresis curves to be very pinched due to softness of the nails and the gap formed by crushing of the wood. Due to the large surface area required to accommodate the large number of nails needed, this type of connection is only suitable for deep, slender glulam members. Finally, nails are used as main connectors in timber shear walls, the main lateral load resisting system in platform construction. Numerous static, cyclic and shake table tests on nailed shear walls showed a large energy dissipating capacity of these structural elements when properly designed.
2.5.2.2 **Dowels**

Dowels are well known connectors that can be used effectively for connecting sawn lumber, glulam or PSL elements. The yielding capacity of the dowel in combination with the bearing strength of the surrounding wood can provide high ductility and excellent energy dissipation characteristics under cyclic loads. Test results on the cyclic behaviour showed that best results are obtained with slender dowels and a relatively large spacing between them. Joints with stocky dowels or small spacing tend to fail in a brittle mode, before large deformations are reached. It was also found that the shape of the hysteresis loops of dowel connections changes progressively with increased displacement (Figure 2.6).

![Hysteresis Loops](image)

*Figure 2.6. Typical hysteresis loops of a dowel connection at different load levels a, b and c (Ceccotti 1996).*

Dowels can be used with a steel shear plate inside the member in the case of an axial or a moment resisting connections (Prion et al. 1994). These aesthetically pleasing connections, showed to be very effective in construction of moment resisting frames in high risk seismic zones as well (Frenette et al. 1996). A recent study on fire resistance of connections showed that
dowel connections with embedded type steel gussets did not need additional protection to satisfy the fire regulations (Komatsu et al. 1990). Recently, much success was achieved in Europe in construction of high capacity connections using large numbers of small diameter dowels (Aasheim, 1994). In addition, a large research project was carried out at the Swiss Federal Institute of Technology in Zurich to optimize the design of high efficiency dowel connections for use in sawn lumber, glulam and parallel strand lumber (Mischler, 1998). This connection is a multiple shear dowel connection with several slotted-in steel plates and is very efficient for large timber members. The results showed that to obtain ductile behaviour in joints with multiple dowels the dowel strength and spacing has to be adapted to the timber properties.

Extensive testing was also performed on a variety of improved dowel-type connections. For example, Rodd (1988) explored the possibility of increased friction around the dowels to improve the embedment strength and avoid splitting of the wood. In other studies, to prevent the occurrence of cracks at the interface of the jointed section where the highest loads are expected, reinforcement was glued to all timber sections separately. Steel plate and fiberglass were examined (Leijten, 1988), however, not with much success. The strengthening effect of fiberglass was found insufficient, while the steel plates, though very effective, were regarded not suitable for practical application. Finally, strengthening with a densified veneer wood (DVW) was studied on a dowel-type connector with expanded tubes (Leijten, 1996). Due to the ability of the tube to absorb the deformation energy, connections showed surprisingly good ductility performance. Similarly, a hollow dowel fastener injected with resin was tested and showed sufficient strength and relatively high ductility capacity (Guan and Rodd, 1996).
2.5.2.3 **Bolts**

Static and cyclic tests on bolted connections showed that the behaviour is mainly dependent on the size of the bolts used. Small diameter bolts tend to behave more like nails or dowels as their shaft can bend without inducing wood fracture, thus exhibiting ductile behaviour and relatively large energy dissipation (Figure 2.7. a). When the slender bolts yield, they also allow for better redistribution of the load among all the fasteners, resulting in improved load sharing. When larger diameter bolts are used, the inelastic behaviour and energy dissipation of the connection depends on the embedding behaviour of the wood alone (Figure 2.7. b), which often leads to brittle failures. Oversized holes and large fabrication tolerances can cause non-uniform load distribution and precipitate fracture of the wood. In this case, the lack of ductility or overloading of one individual bolt can lead to the initiation of a fracture that precipitates through the group (Ceccotti, 1996). Bolted connections are probably the most frequently used connections for heavy timber frames, and in particular for axially loaded braces and struts. It is for this reason that several different bolted connections were investigated in this thesis.

![Figure 2.7. Typical hysteretic behaviour of connections with a) slender and b) stocky bolts.](image)

Figure 2.7. Typical hysteretic behaviour of connections with a) slender and b) stocky bolts.
Numerous studies were carried out to determine the factors that influence the strength and behaviour of multiple-bolted connections. Parameters such as wood and bolt properties, end distance, edge distance and fastener spacing, both parallel and perpendicular to the grain were investigated. Trayer (1932) presented design formulae for bolted joints with steel side plates based on test data (Moss, 1997). He was the first to note that the interaction of the bending of the bolt and the crushing of the wood affected the joint performance. In 1949, Johansen introduced a theory to predict the ultimate strength of a bolted (dowel) type joint based on equilibrium equations derived from a free body diagram of a bolt in a wood member. The European Yield Model, based on Johansen’s theory and subsequently refined by Larsen (1973), is currently used in many timber design codes in the world. Larsen’s prediction equations assumed that a bolted connection could fail by full bearing (crushing) in one or more wood members (for short bolts), or by yielding of the bolt (for longer bolts) accompanied by local wood crushing near the member interfaces. The model considers several possible failure modes for a given joint configuration, based on the relative strength between the connector and the wood. Figure 2.8 shows the different failure modes assumed in the European Yield Model for a three-member bolted joint.

![Figure 2.8. Failure modes assumed in European Yield Model for three-member joint.](image-url)
Kunesh and Johnson (1968) investigated the strength of multiple-bolted joints as a function of spacing, seasoning, and types of loading. They found that joints fabricated with seasoned material, but tested dry, had a significantly higher bearing capacity than those fabricated with unseasoned material. In addition, they observed that the staggered bolt joints had a greater load-carrying capacity than the regular-pattern joints. Yasamura (1987) investigated the influence of end distance, fastener spacing and number of bolts on the ultimate properties of bolted joints in glued-laminated timber. He found that an increase in the ultimate load per bolt proportionally to the increase of the bolt slenderness. Another trend observed was that as the number of bolts in the connection increased the ultimate load per bolt decreased. The test results also showed that longer bolts showed greater load reduction with reduction of bolt spacing.

A study at University of British Columbia (UBC) has shown that the ductility and resistance of a single bolted connection in Parallam can be enhanced significantly by adding reinforcement in the form of a truss plate, fiberglass, or a layer of glued-on plywood (Prion, 1998). For multiple-bolt connections, however, only a slight increase in strength was achieved due to better load distribution among the bolts. The most important gain observed was the vastly improved ductility and much higher consistency in the connection behaviour. Another simple and inexpensive way to reinforce a bolted connection is to insert reinforcing rods perpendicular to grain and to the bolts. The reinforcement could be done with either glued rods with epoxy or with threaded rods driven into tight fitting holes. A study at UBC with various reinforcement patterns showed that when reinforcement bars were added halfway between the bolts, a significant improvement was achieved in both strength and ductility of the connections (Prion, 1998).
2.5.2.4 Glulam Rivets

Glulam rivets, also known as Griplam nails or timber rivets, are high strength nail-type fasteners developed in Canada especially for use in glued-laminated timber construction. Over the years, they have become the favoured fasteners for glulam because of numerous advantages over other connectors. They are stiffer and provide greater load transfer per unit contact area than any other wood fastener. The wood members need not be drilled or grooved which simplifies the fabrication and the field assembly. Although they are made of high strength steel with high yield point, they are soft enough to provide very ductile connections. Figure 2.9 shows a typical column to beam glulam riveted connection.

![Figure 2.9. Typical column to beam glulam riveted connection.](image)

The development of glulam rivets dates back to the 1960s when numerous rigid-plate joints utilizing over 40,000 rivets were examined by Madsen and McGowan (Williams and
Karacabeyli, 1996). Design of glulam rivet connections in the current edition of the Canadian Code for Engineering Design in Wood (CSA-O86.1-M94) is based primarily on the method developed by Foschi and Longworth (1975). According to this method, the ultimate capacity of the connection is governed by either a rivet yielding or a wood failure mode. The rivet yielding failure mode, where the rivets bend and yield while the wood under their shanks fails in crushing, was first studied by Foschi (1974). Using finite element analysis, he also studied the effect of rivet penetration and the effect of direction of loading with respect to grain orientation. The load carrying capacity of the glulam riveted connections in rivet yielding mode can also be determined using the European Yield Model (Karacabeyli et al. 1994; Buchanan and Lai, 1994).

For the wood failure mode, Foschi and Longworth (1975) investigated the stress distribution around the rivet cluster and derived some formulae for calculating maximum stresses in the member around the rivets. Weibull’s weakest link theory was used to determine the ultimate wood strength. They verified their analytical model with test results on riveted connections in Douglas-fir glulam and offered the following conclusions: (i) rivet spacing controls the failure mode; (ii) larger spacing results in a rivet yielding failure mode while a smaller spacing produces sudden wood failures usually a block-shear around the group of rivets; and (iii) for the same rivet spacing, a larger end distance leads to an increase in the ultimate load based on the wood shear failure.

The research results by Foschi were complemented by further experimental studies. Fox and Lincoln (1979) investigated the effect of plate thickness and hole size on the rivet yielding capacity. Karacabeyli and Foschi (1987) performed a theoretical and experimental study on eccentrically loaded glulam riveted connections. They developed a simplified model for
predicting the load carrying capacity for the rivet yielding mode and made recommendations to avoid a wood failure mode in moment connections. Karacabeyli and Fraser (1990) carried out experiments to extend the application of glulam riveted connections to spruce-pine glulam. Finally, Karacabeyli et al. (1994) carried out a research study on the use of riveted connections in sawn lumber. They also investigated the effect of plate thickness on the joint capacity and determined the withdrawal strength of the rivets.

2.6 BRACED FRAME SYSTEMS IN TIMBER CONSTRUCTION

For large nonresidential buildings or for bridges, a braced or triangulated structural system is often the best solution to resist lateral loads induced by earthquake forces. It allows for flexibility in design and uses wood in its strongest direction - parallel to grain in tension or compression. A braced frame system can be used in glued-laminated, post and beam or in conventional wooden construction (Fig. 2.10).

Figure 2.10. Braced timber frame in residential construction in Japan (photos by H.G.L. Prion)
Different types of braced timber frames are shown in Figure 2.11. Diagonal braces (single or X braces) have the disadvantage of obstructing movement of people and goods and are thus usually located in the plane of the wall. An alternative solution is to use knee-braced frames, which are widely used in industrial structures (Fig 2.11.c). These frames consist of short, inclined members fitted into the upper corners, where columns and beams meet. The addition of knee bracing to a beam and column assembly essentially creates a frame with moment resistant beam to column joints and is a particularly effective and simple method of achieving lateral resistance without obstructing passage. In this case, however, significant bending and shear forces are introduced in the members, resulting in a more flexible system. To avoid tensile stresses across the grain, careful detailing of the connections is required.

In concentrically braced frames (single or X braced) eccentricity in the joints is minimized and lateral forces are resisted by almost pure axial loads in the brace members. If single diagonals (concentrically trussed frames) are used, they have to serve a dual function: acting in tension for the lateral loads in one direction and in compression for a load reversal. Wooden braces are
more economical in this case since the capacity is mainly governed by the buckling resistance, for which the larger wood cross-sections are particularly suited. A combination of wood and steel members can also be used to gain the most from both materials.

Because long bracing members are more efficient in tension than in compression, frames are often braced with a crisscrossed set of diagonals, which act in tension only. In this case it is usually more economical to use steel braces. If the bracing members are designed to resist axial stresses only, it is important that they be attached in a way so that secondary moments are not generated through a joint rotation. In steel braces, a substantial amount of energy can be absorbed by means of brace yielding, provided that the connections are designed properly to avoid wood failures. In unidirectional loading, stretching of the tensile braces and buckling of the compression braces occurs. Because the braces are almost certain to be in the slender category, it is very unlikely that compression yielding will occur, so the compression braces will become largely ineffective as they buckle. When the direction of load reverses, the previously yielded braces will buckle relatively early and the other braces, now in tension, will yield. After a few cycles, damage often occurs during the compression cycle, followed by a fracture during the next tension cycle. Generally, in the case of steel X braces the response of the frame during subsequent cycles of loading is strongly affected by the fact that both braces have been stretched. The stiffness of the frame near the original equilibrium position is now greatly reduced and damage due to momentary acceleration of the structure (also called whipping) may cause very high impact forces, leading to serious damage or collapse.
In most single-storey multi-bay frames, it is not necessary to brace every individual bay of the frame system. In fact, this is often not possible for architectural reasons, so usually only a few bays are braced (Figure 2.12.a). Similarly, multi-bay multi-storey structures must have at least one bay fully braced along the height of the structure (Figure 2.12.b). One important lesson learned from previous earthquakes was that the redundancy of the resisting systems is very important, suggesting that only one set of vertical bracing should not be considered adequate for buildings in high risk earthquake zones. Braced timber frames can also be combined with other lateral load resistant systems, provided that compatibility requirements are met. An example of the use of a braced frame to resist lateral forces in one direction and a set of timber shear walls in the other direction is presented in Fig. 2.13. The two systems act independently, except in the case of torsion, in which case the load sharing is not equal due to a difference in the stiffness of the two systems.
2.6.1 Recent Studies on Braced Timber Frames

A limited amount of information is available on the seismic behaviour of braced timber frames. Series of tests on one storey, single braced timber frames were performed at the University of Canterbury, New Zealand by Deam (Buchanan, 1989). The main objective of the research project was to determine the seismic loading properties of the frames, with emphasis on the ductility capacity of the frames, so they can be designed for reduced lateral loads. Braced frames with both steel and wood diagonals were considered. The ductility properties of the frames were found to be largely dependent on the behaviour of joints (Buchanan et. al 1989). In joints where compression stresses perpendicular to grain were developed, relatively large ductilities were achieved, while poor behaviour was observed in the frames with brittle or weak connections and in frames where the connection strength exceeded the tensile strength of the wood members.
Three different types of connectors were used in the study: nail-plate, tooth-plate, and bolted connections. Nail-plate connections were found to be very ductile, especially when splitting of the wood was prevented. Tooth-plate connectors exhibited relatively brittle performance and often tore across tooth-lines. In some cases, bolted joints were found to be weaker than the design strengths, mostly because of premature splitting of the wood. The tests demonstrated that timber joints in general are capable of developing high ductilities if splitting of the wood is prevented and if the connector itself can sustain large cyclic deformations. Nail-plates were found to best fulfill both of these requirements. Finally, ductility of the connections at the base was found very important for the behaviour of the frame.
Another study, which included a series of tests on timber braced frames and an analytical investigations, was conducted at the Department of Structural Engineering, Building Research Institute, Tsukuba, Japan (Yasumura, 1990). A total of seven full scale, trussed braced frames of glued-laminated timber were subjected to reversed cyclic lateral loads. The specimens with wooden braces and bolted connections were designed to represent a single storey building of 7.5 meters height. To avoid the effect of brittle failure at the joints due to rotation of the brace, six out of seven specimens had bolted connections with steel pin joints at the end. To compare the effects of the joint fixity, the seventh specimen had bolted steel side plates, directly connected to the frame. The pin-connected specimens were divided into two groups: three with inserted steel plates inside the wood member and three with steel side plates. The ratio of the thickness of the braced members to the diameter of the bolt was four, eight and twelve. A typical hysteresis loop of the force deformation relationship, obtained during the testing is shown on Figure 2.15.

![Typical hysteresis loop obtained during the testing (Yasumura, 1990).](image-url)
The tests showed that all the specimens failed with the destruction of the end joints of the brace members due to shear failure or splitting of the wood. In some specimens, partial failure of wood occurred along the bolt line after yielding and cracks spread as the load was increased. The braces failed in tension in all the specimens. Fixed end joints caused considerable stresses perpendicular to grain due to rotation of the brace. In the specimens with steel pin joints, a rotation of the connector plates was observed in the compression braces. The load displacement curves showed the non-linear hysteresis loops with initial slip due to embedding of the bolts into the wood. Strength degradation after substantial yielding of the bolts was observed in some specimens.

For dynamic loading conditions, equivalent viscous damping was found to be dependent on the lateral displacement of the frames. It showed values of approximately 15% for horizontal displacements less than 1/200 of the frame height, and decreased to approximately 10% when horizontal displacements were approximately 1/100 of the height. One of the major observations from the study was that the performance of the frames improved when the brace member thickness to bolt diameter ratio was equal to or more than eight. Stresses perpendicular to grain due to rotation of the brace at the end joints should be avoided by means of a steel pin joint or other equivalent method. In the analytical part of the study, load displacement hysteresis loops obtained experimentally for the entire frames, were modelled with a bilinear slip model. Time-history response analyses of a single degree of freedom system were carried out to investigate the influence of the yield load level on the structural response. Based on the time history analysis with three different accelerograms, the ductility factor (force modification factor) was estimated to be 2.0 for ductile braced frames and 1.5 for non-ductile frames.
2.7 SEISMIC DESIGN CONSIDERATIONS

The fundamental aim of earthquake-resistant design of buildings is to provide structures with an acceptable level of safety for public use. This is achieved by specifying design loads and detailing requirements, so that the probability of building collapse or injury to people is acceptably low when the structure is subjected to a certain level of ground motion. Code requirements for achieving seismic resistant buildings have evolved in a semi-empirical manner by simplified mathematical and experimental models based on the experience of the structural behaviour in previous earthquakes.

The occurrence of a major destructive earthquake is usually rare during the lifetime of a structure. Therefore, the seismic provisions usually do not require all buildings to have the necessary reserve strength and capacity to withstand the largest possible earthquake without suffering any damage, which is also economically not viable. For example, the National Building Code of Canada (NBCC, 1995) aims to minimise the probability of injury and loss of life by adopting the philosophy that structures should be designed: (a) to withstand a minor earthquake without any damage; (b) to resist a moderate earthquake without significant structural damage, but possibly with some non-structural damage and (c) to resist a severe earthquake without collapse or major failure.

2.7.1 Capacity Design Philosophy

In the present seismic design philosophy of timber structures, the concept of capacity design is of major importance. This design approach, first introduced in New Zealand, is based on the
simple understanding of the way a structure sustains large deformations under severe earthquakes. By choosing certain modes of deformation, we can ensure that the brittle elements have the capacity to remain intact, while inelastic deformations occur in selected ductile elements. These "fuses" or energy absorbers act as dampers to reduce force level in the structure (Prion, 1998). It is also very important to have these ductile locations distributed in the structure at strategic locations so that the vital parts of the lateral resistant system are not destroyed.

In steel structures where the ductility of the connections is often questionable, the members are typically designed to yield before the connections. Beam failure mechanisms are preferred since they provide sufficient structural ductility without creating a mechanism of collapse. In timber structures, however, the failure of wood members in tension or bending is not favourable because of its brittle characteristics. The potential of the wood to absorb energy in compression perpendicular to grain has to be exploited and ductile connectors that are weaker than the wood members have to be used. Since earthquakes induce numerous reversing load cycles on the structure, it is important to emphasize not only the ductility achieved during the first load cycle but also the behaviour during the subsequent cycles. Another important aspect in seismic timber design is to assure a complete load path in the structure with high level of redundancy. The connections that are expected to deform significantly have to be identified and designed in a highly ductile manner. Their eventual failure should not result in a structural collapse.

2.7.2 Methods of Analysis

In seismic design of structures, different approaches can be followed to achieve the desired structural performance. A complex approach involves a complete time history dynamic analysis
of the structure under several probable earthquakes. The complete response history analysis (RHA) with information on peak response, time of peak occurrence, and number of excursions over a specified response level can therefore be obtained. In this case, each structural element is modelled by a non-linear hysteretic model that adequately represents the load deformation properties of that element. A number of hysteresis models for different timber structural components were developed over the years (Dowrick, 1986; Foliente, 1997). For most structures, a response spectrum analysis (RSA) can also provide the information required for the design. Based on the estimates of mass and elastic stiffness properties of the structure, the RSA procedure allows for computation of the peak response values during an earthquake, represented by its response spectrum. It reduces the dynamic problem to a series of static analyses, taking into account the initial vibrational properties and damping ratios of the structure as well as characteristics of the ground motion.

The most common design approach is to convert the dynamic loads on buildings to equivalent static lateral loads. This method is often used in seismic design codes around the world. Buildings are designed to resist specified equivalent static forces related to the properties of the structure and seismicity of the region. Based on the estimate of the fundamental period of vibration of the building, formulae are specified for the total base shear and distribution of lateral forces over the height of the building. These forces do not represent the peak dynamic forces that may be exerted on the structure during a typical earthquake for that site. It is only expected that a structure designed to resist these equivalent static loads will perform satisfactory in accordance with the design objectives. The total lateral load $V$ on the building according to the National Building Code of Canada (NBCC, 1995) is calculated according to the formula:
\[ V = \frac{V_e}{R} \cdot U \] 

where: \( V_e \) - Equivalent lateral seismic force representing elastic response;

\( R \) - Force modification factor (\( R > 1.0 \));

\( U \) - Calibration over-strength factor (\( U = 0.6 \)).

The equivalent elastic seismic force \( V_e \) is calculated as a product of the zonal velocity ratio \( v \), the seismic response factor \( S \), the importance factor \( I \), foundation factor \( F \) and estimated weight of the structure \( W \). The seismic response factor \( S \), represents an idealized elastic multi-degree-of-freedom structural response with 5% damping, for unit values of zonal velocity ratio \( v \) and weight \( W \). The values for \( S \) depend on the period of the structure and the seismic region where the structure is located, as shown in Figure 2.16.

![Figure 2.16. Seismic response factor S in the 1995 version of NBCC.](image)

As shown in equation 2.1, the design force specified in the NBCC is reduced from the level that corresponds to fully elastic structural response by a force modification factor or so-called \( R- \)
factor. Different \( R \)-factors are assigned to different types of structural systems reflecting their design, construction experience and performance during past earthquakes. The factor accounts for the energy absorption capacity of the structural system through damping and inelastic action during load reversals. Those types of structures that performed well during past earthquakes are assigned higher values of \( R \)-factor in NBCC. The calibration factor \( U \) is based mainly on experience and accounts for the building over-strength due to material over-strength, structural redundancy and some not-accounted-for elements such as cladding and partitions.

2.7.3 Force Modification Factors

As mentioned before, a force modification factor accounts for the capability of a structure to absorb energy within acceptable deformations and without failure, and thereby reduce the structural response. In addition, it takes into account the existence of alternate load paths and redundancy of the structural system. If there are more locations in the structure where energy can be dissipated, the risk of collapse is reduced when some individual member fails or becomes severely damaged. A building designed with a value of \( R \) greater than 1.0 is presumed capable of undergoing inelastic cyclic deformations (Figure 2.17).

There is very little theoretical or experimental background for the numerical values of the currently assigned \( R \) factors in the codes. The effects of other parameters in addition to general building type are not quantified, so the definition of \( R \)-factors requires considerable individual judgement. Consequently, the values of \( R \)-factors given in different codes vary significantly. Recent studies have shown that the reduction of the elastic spectrum (\( R \)-factor) is a function of many different parameters. Some parameters include the initial period of the structure, ductility
factor or target ductility, the shape of the load deformation relationship, the degree of damping, the level of over-strength of the structure, characteristics of ground motion and of the $P-\Delta$ effects. In the case of multi-degree-of-freedom (MDOF) systems, failure mechanism, torsional effects, vertical strength and stiffness irregularities as well as soil-structure interaction also affect the force modification factors. From all these parameters, ductility and over-strength appear to have the largest impact on $R$ values.

![Figure 2.17. Force-deformation relationship for simple elastic-perfectly plastic systems.](image)

The dependence of the force modification factors on the initial period of the structure is important for the design of relatively rigid structures, which have fundamental periods of vibration of 0.5 sec or less. This range represents the most of low to medium rise timber structures, especially the shear wall type ones. Period dependence is also important when considering the higher mode effects on more flexible structures. For reduction of the linear elastic spectral values for intermediate and long period buildings (equal displacement range),
there is a widely accepted approach to reduce these values by a factor $\mu$, (so $R=\mu$), where $\mu$ is the displacement ductility factor for a single degree of freedom system (Figure 2.18).

Figure 2.18. Influence of structural period on ductile force reduction (Paulay & Priestley, 1992).

For short period structures, on the other hand, there is little agreement on recommended $R$ values. The proposition to reduce the elastic spectral values by a constant factor of $\sqrt{2\mu - 1}$ in the intermediate and short period ranges (equal energy range) has been widely used, in spite of the fact that the relation between the elastic and inelastic spectrum is not constant. This reduction can only be justified if the structure is subjected to relatively short acceleration pulses with respect to the fundamental period of the structure, or when the input energy for the linear elastic structure is the same as that for an equivalent inelastic (perfectly plastic) structure (Bertero, 1986). Unfortunately, these assumptions are not realistic for most cases of seismic response.

Paulay and Priestley (1992) suggested that for long period structures ($T > 0.7$ sec) the relation $R=\mu$ is a reasonable assumption. At $T = 0$ and very short periods, they suggested a factor of $R =$
1.0 regardless of ductility \( \mu \). Between \( T = 0 \) sec and \( T = 0.7 \) sec, a linear increase in \( R \) factor as a function of \( T \) can be assumed, according to the relationship:

\[
R = 1 + (\mu - 1) \frac{T}{0.7}
\]  

(2.2)

Not many codes or design recommendations so far have adopted force modification factors that depend on the natural period, although their dependence on the structural period as described above was acknowledged for years. One of the few codes that recognizes the influence of the initial period of the structure on the force modification factors is the Mexican Code. It specifies a reduction factor that decreases linearly with the period of the structure in the short period range. (Hidalgo et. al. 1990). According to the code, for structures classified as regular, the earthquake forces determined by the static method may be reduced by a factor \( Q' \), which is determined as follows:

\[
Q' = 1 + \left( \frac{T}{T_a} \right) (Q - 1) \quad \text{for} \quad T < T_a \]  

(2.3)

where:

- \( Q \) - Factor that depends on the type and characteristics of the structure;
- \( T \) - Fundamental natural period of the structure;
- \( T_a \) - A period from Code design spectrum that depends on seismic zone.

For structures with fundamental natural period \( T > T_a \), the reduction factor \( Q' = Q \). A similar approach for short-period structures can be found in The Venezuelan Seismic Code of 1987 (CONVEIN 1756-87). Period dependent \( R \)-factors are also considered in the draft of the Chilean Code, NCh433.cR89 (Hidalgo et al. 1990). According to the code the response modification factor is defined by the following equation:
where:

\( T \) - Initial period of the structure;

\( T_0 \) - Factor dependent on the soil conditions;

\( R_0 \) - Factor dependent on the structural ductility and observed structural behavior.

To incorporate the effects of over-strength and ductility of the structure in the R factor, Fischinger, Fajfar and Vidic (1994) defined the R-factor as a product of the equivalent global ductility factor \( R_M \) and the over-strength factor \( R_s \) as follows:

\[
R = R_M \cdot R_s
\]  

(2.5)

Although this relationship is simple, the quantification of these factors is very complex. The authors proposed explicit relations for the factors that depend on the period of the structure, transition period in the spectrum, maximum ground velocity and acceleration, corresponding spectral amplification factors as well as on several constants that describe the hysteresis behaviour and damping.

Riddell and his collaborators (1989) defined the response reduction factor \( R \) from the average spectral acceleration for different ductility levels. They used four sets of records and spectra obtained for elastic and inelastic systems for various ductility levels. They calculated the R factor for each set of records according to the following formula:

\[
R = R (T, \mu) = \frac{S_a(\mu = 1)}{S_a(\mu)}
\]  

(2.6)
where: \( S_a(\mu = 1) \) - Acceleration response spectra for linear system (ductility = 1),
\[ S_a(\mu) \] - Acceleration response spectra for inelastic system with ductility \( \mu \).

They found that the R-factor curves approached a value very close to 1.0 for very short period structures and levelled off at a constant value larger than 1.0 for periods over 0.5 seconds.

In the field of timber structures, Ceccotti and Vignoli (Ceccotti et al. 1988) suggested a behaviour factor \( q \) (European version of R factor), as the ratio between the collapse acceleration \( (A_u) \) and the acceleration that produces the elastic limit stress \( (A_y) \):

\[
q = q(T, \mu) = \frac{A_u}{A_y}
\]  \hspace{1cm} (2.7)

The equation 2.7 reads that parts of the structure reach the yield point when subjected to acceleration \( A_y \), while the structure reaches collapse point when accelerated by \( A_u \). The authors numerically investigated two types of simple glue-laminated portal frames, one base restrained \( (T = 0.26 \text{ sec}) \) and the other base hinged \( (T = 0.52 \text{ sec}) \). In addition, two different types of joints were studied: rigid glued joints and semi-rigid joints with dowels connectors. Series of experimental tests were previously conducted at the Civil Engineering Department, University of Florence, Italy, to assess the behaviour of the connections. The moment-rotation relationship of the joints was modelled using a bilinear law. A non-linear dynamic analysis computer program (DRAIN-2D), which uses the direct integration method, was used for the analysis. Six different accelerograms were used as base excitation of the portals, two of them real records (Tolmezzo N-S and Tolmezzo SE-NW from the 1976, Friuli earthquake) and four artificial records (GEN1, GEN2, GEN3 and GEN4) obtained using the SIMQKE program (Vanmarcke, 1976, Department of Civil Engineering, Cambridge, Massachusetts).
The maximum acceleration values ($A_y$) that correspond to the elastic limit, and the maximum acceleration value ($A_u$) that causes collapse, were assessed for each structural scheme, each type of connection and for each accelerogram. In the case of structures with rigid joints, given the brittle behaviour of the wood, the failure mechanism occurred in the section with maximum stress. In the case of semi-rigid joints, the collapse typically took place when a section breaks down and when the beam displacement exceeds the pre set value of $1/20^{th}$ of the portal height. The main results concerning the structural behaviour factor $q$, are shown in Figure 2.19.

![Figure 2.19. Behaviour factor as a function of the predominant frequency (Ceccotti et al. 1988).](image)

The analysis showed that in case of semi-rigid joints (nails or dowels), the structural behaviour factor $q$ can be ten times larger than in case of glued joints. Although these results are just an approximate indication of the structural behaviour values for timber structures, they stress once again the importance of having ductile connections in timber structures.
In more recent study, Karacabeyli and Ceccotti (1998) carried out a verification of the force modification factor $R$ used for timber shear walls in the National Building Code of Canada. While initially there was no adequate information on this factor, they have shown that $R=3.0$ is a realistic value, describing the energy dissipating properties of a multi-storey timber shear wall building.

The general theoretical and experimental knowledge on seismic behaviour of timber structures, however, is lagging behind that of steel or concrete structures. Much work still needs to be done to characterise the behaviour of different timber framing systems and obtain realistic and consistent force reduction factors. In the area of the braced timber frames, there is virtually no available information and the field is wide open to experimental and analytical investigation. Therefore, one of the main objectives of the research presented in the following chapters of this thesis is to narrow the gap of knowledge with some new information on seismic behaviour and force modification factors for braced timber frames.
3. QUASI-STATIC CONNECTION TESTS

Quasi-static testing is the most common experimental procedure in structural and earthquake engineering because of its relative simplicity and cost effectiveness when compared to other methods. Much of the current knowledge on seismic performance of timber connections and components was derived from simple quasi-static testing. The term quasi-static testing (QST) indicates that the loads are applied at rates slow enough, so that the material strain rate effects do not influence the results. Although, QST refers to either monotonic or cyclic tests, the later is more commonly used in earthquake related research. Conventional pseudo-dynamic tests at very slow rates are also considered quasi-static. The main benefit of quasi-static testing is that it allows close monitoring of the connection behaviour under controlled loading conditions. The response obtained from QST, unlike interpretations from purely analytical research, consists of all possible failure mechanisms whether anticipated by the researcher prior to the test or not.

The findings from quasi-static tests on connections commonly used in braced timber frames are presented in this chapter. The quasi-static tests represent the first part of the experimental program presented in this thesis. The rest of the experimental program, which includes shake table tests on single brace frames and shake table tests on a two-storey braced frame model, are presented in the subsequent chapters.
3.1 OBJECTIVES AND SCOPE

Since the seismic response of braced timber frames largely depends on the connections between the braces and the rest of the frame, the main objective in the first part of the experimental program was focused on characterizing the seismic behaviour and failure modes of different connections used in braced timber frames, when subjected to a standard cyclic loading protocol. This is a necessary first step toward developing an analytical model for prediction of the connection behaviour, and subsequently predicting the behaviour of a braced frame system under dynamic loads. Displacement controlled monotonic and cyclic tests were conducted on a number of connections with four different connector types, namely mild steel bolts of 9.5, 12.7 and 19 mm diameter, and glulam rivets. The quasi-static connection tests were conducted in the Wood Engineering Laboratory of Forintek Canada Corp. in Vancouver.

3.2 METHODS AND MATERIALS

3.2.1 Brace Connection Specimens

Test specimens were chosen to represent typical connection details used in braced timber frames (Figure 3.1). It was decided to limit the testing program to the most commonly used connections, which use external steel plates, connected to wood brace members with bolts or glulam rivets. As shown in Figure 3.1, the bolted connections are loaded in double shear, while the riveted connections are loaded in single shear. As discussed in the previous chapter, glulam rivets are high-strength, oval shaped nails with a tapered head. The flat cross section of the shank prevents splitting of the wood while the tapered head, after driven into an undersized
round hole in the steel plate, creates a wedging action that gives fixity to the rivet head. The wedging also increases the lateral stiffness of the rivet, which acts as a cantilevered beam on an elastic foundation, and ensures formation of a plastic hinge at the head of the rivet under large deformations. A typical glulam rivet and steel plate used for one of the riveted connections tested is given in Figure 3.2.

![Figure 3.1. Typical details of riveted and bolted braced frame connections.](image)

The brace specimens consisted of grade C, Spruce-Pine-Fir (SPF) glued-laminated timber, 130 x 152 mm (5" x 6") in cross section. Steel plates for bolted connections were 12 mm (1/2") thick, while riveted connections were built using 6 mm (1/4") plates (Figure 3.2). Mild steel ASTM A307 bolts of three different diameters were used: 9.5 mm (3/8"), 12.7 mm (1/2") and 19 mm (3/4"). The ASTM A307 bolt is comprised of low carbon steel having a minimal tensile strength of 414 MPa (60000 psi) for bolt dimensions from 6.4 mm to 38.1 mm. (ASTM, 1993). The member thickness to bolt diameter ratios (bolt slenderness) were 13.3, 10.0 and 6.7, respectively. For glulam riveted connections, 65 mm (2.5") long rivets were used. Six replicates were tested from each connection, three under static and three under cyclic loading conditions,
except for the 12.7 mm bolted connection where the specimens from the preliminary tests were included as the fourth replicate.

![Diagram of Section A-A and Section B-B, showing 1/4" bolt and 1/8" rivet, with circular holes and a 1/4" steel plate.]

Figure 3.2. A glulam rivet and steel plate hole arrangement of a riveted connection.

The design load (factored resistance) for bolted connections was calculated according to the Canadian Standards Association - Engineering Design in Wood (CSA-O86.1-94), using dry service conditions ($K_{sf} = 1.0$), and assuming short term loading ($K_d = 1.15$). The same applied for the riveted connections except that the load duration factor $K_d$, according to CSA-O86.1, was not included when calculating the factored resistance for short term loads in rivet yielding mode. The end distance $e$ for all bolted connections was 12 times the bolt diameter (12d), which is larger than the minimum specified distance of 10d according to CSA-O86.1. The spacing $S$ between bolts in the row was 4d, which is the minimum requirement, while the spacing between the rows $s_r$ was 3d. The spacing between rivets in the riveted connections ($S$ and $S_r$) was 25 mm in all directions, while the end distance $e$ was 75 mm.
Table 3.1. Configuration and specifications for the tested connections.

<table>
<thead>
<tr>
<th>Test Group</th>
<th>Fastener</th>
<th>Design Force (kN)</th>
<th>End Distance e (mm)</th>
<th>Row Spacing $s_r$ (mm)</th>
<th>Connector Spacing $s$ (mm)</th>
<th>Configuration</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>9.5 mm Bolts</td>
<td>67.6</td>
<td>114</td>
<td>30</td>
<td>38</td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>12.7 mm Bolts</td>
<td>64.4</td>
<td>152</td>
<td>40</td>
<td>51</td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>19 mm Bolts</td>
<td>56.9</td>
<td>228</td>
<td>-</td>
<td>76</td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>Glulam Rivets</td>
<td>90.7</td>
<td>75</td>
<td>25</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td>II</td>
<td>9.5 mm Bolts</td>
<td>28.1</td>
<td>114</td>
<td>-</td>
<td>38</td>
<td></td>
</tr>
<tr>
<td>II</td>
<td>12.7 mm Bolts</td>
<td>24.2</td>
<td>152</td>
<td>-</td>
<td>51</td>
<td></td>
</tr>
<tr>
<td>II</td>
<td>Glulam Rivets</td>
<td>34.0</td>
<td>75</td>
<td>25</td>
<td>25</td>
<td></td>
</tr>
</tbody>
</table>

The selected connections were divided into two test groups, based on their design load (factored resistance) calculated according CSA086. The first group consisted of connections that have a design load around 70 kN (from 57 kN to 91 kN), which was deemed to be a realistic load level not to cause buckling of the brace element chosen. The second group consisted of connections with design loads of around 30 kN (from 24 kN to 34 kN). Reduced design levels for the second connection group had to be used due to the limited load capacity of the shake table, where these connections were used for dynamic tests on single braced frames, later in the project. In addition, all bolted connections from the second group had only one row of bolts compared to the connections in the first test group, which had two rows of bolts, except for the 19 mm
connection. By comparing the results from both groups, the influence of the number of rows on the design load and behaviour of the connections could be investigated. Specifications of the connections tested are given in Table 3.1.

The glulam members were conditioned prior to testing in a laboratory environment at an average temperature of $20 \pm 3^\circ\text{C}$ and relative humidity of $50\% \pm 10\%$ for six months, which characterises typical in-service conditions. Bolted connections were fabricated within a maximum of 72 hours prior to testing. Bolt holes in the wood members and the steel plates had the same diameter as the bolts, although commonly used assembly tolerances allow for oversizing of up to 1.6 mm (1/16 inch). The members were carefully bored using a radial-arm drill, so that the holes were perpendicular to the surface of the member. The surface of the hole was made smooth and uniform to assure good bearing of the bolts. Accurate centring of the holes was required in connections with multiple rows of bolts.

Riveted connections were fabricated by hand, driving the rivets with a hammer in pre-drilled 6.8 mm circular holes in the steel plates. In order to avoid splitting of the wood, each rivet was placed with its major cross-sectional dimension aligned parallel to the grain. Glulam rivets at the perimeter of the group were driven first, while the successive rivets were driven in spiral pattern from the outside to the center of the group. After fabrication, riveted connections were conditioned in a dry laboratory environment for a minimum of three weeks to allow for the relaxation of wood fibres around the rivets. This conditioning time provides a more accurate representation of a riveted connection that has been in service for a longer period. Riveted connections usually have a higher initial stiffness immediately after assembly because the wood fibres in contact with the rivets have not yet relaxed.
3.2.2 Testing Apparatus and Instrumentation

A simplified scheme of the test setup for quasi-static connection tests is shown in Figure 3.3. The glulam brace specimens with length of approximately 1.5 m were held in place at the bottom by an over-designed hold-down bolted connection using eight 22 mm (7/8 inch) bolts. The connection to be tested was always on the top of the specimen and its steel side plates were the active members of the test setup. They were connected to a bolted fixture attached to the load cell and an MTS servo-controlled actuator. Two rotational hinges (pins), one at the top and one at the bottom, were introduced to minimise the influence of bending moments and ensure an almost pure axial state of loading for the specimens. A pair of rollers placed on both sides of the specimen prevented it from out-of-plane movements during the compression half cycles.

Figure 3.3. Quasi-static test setup: a) simplified scheme; b) picture of the setup.
Four data measurements were collected during the tests: applied load, movement of the actuator head (stroke), and two relative deformations (slips) of the tested connection. Connection slip was measured using two displacement transducers (DCDT), one on each side of the specimen. Both DCDT transducers had a displacement measuring range of ± 25.4 mm (1 inch). The load was obtained using a 100,000 lbs. (454 kN) load cell attached to the hydraulic actuator. Data was acquired using Forintek's data acquisition software on a personal computer and was analysed using commercial spreadsheet software.

3.2.3 Cyclic Testing Protocol

In order to establish the design properties and evaluate the performance of the connections tested, a monotonic and a cyclic loading regime had to be specified. The issues considered in choosing a standardized test protocol were: (i) the loading regime procedure and (ii) methodology of analyzing the data obtained from the tests. In the field of timber engineering a consensus on the best standard test protocol has not been reached yet. A number of protocols for cyclic testing of timber joints with mechanical fasteners have been proposed by Reyer and Oji, (1991), Ceccotti, (1994), and Foliente, (1994), and their influence and implications on the cyclic performance of the joints has been studied. It was found that test protocol characteristics such as: the amplitude and number of cycles, frequency or velocity during the testing, duration of the test, procedure for defining the yield point and ductility, ultimate load, strength deterioration, and assessment of stiffness and energy dissipation of the joints, appear to be very important. Based on the research results, a number of test standards are under development (CEN 1995, ASTM 1993) or have already become standards (AS 1995).
Although the different standards agree on the protocol parameters that influence the cyclic properties of a connection, they differ in the proposed methods for determining the yield point from the envelope curve of the load-deformation response. A definition of yield deformation, ultimate deformation, and ductility according to the Modified ASTM protocol, Australian, European CEN, and the protocol used in Forintek (FCC), is presented in Figure 3.4. In order to enable comparison of tests data from different test laboratories a standardized test procedure is needed. For that reason, the International Standards Organization's Technical Committee on Timber Structures (ISO TC 165) established a working group (WG7) for the development of an international testing standard. A working draft of this standard was prepared and is available to the researchers since 1998.

Figure 3.4. Definition of yield and ultimate displacements according to different standards.

In Canada and many other countries, there is no established national standard for quasi static testing of timber joints. For that reason, slightly modified FCC testing protocol was adopted here for the quasi-static tests of the brace connections. Monotonic tests were conducted by
applying a tension load to the connection with a displacement rate of 5 mm (0.2 inches) per minute. The adopted cyclic protocol also takes care of all the necessary details that influence the joint performance. Firstly, since seismic excitation of structures is displacement driven, the loading regime used was displacement controlled. Secondly, since tests have revealed that strength degradation occurs when a connection is repeatedly loaded to the same deformation level, three cycles at the same deformation level were considered. In addition, the cyclic loading protocol for each connection was defined in terms of the average yield deformation $\Delta_y$, obtained from the monotonic tension tests on the three replicates for that particular connection. The loading regime in terms of yielding deformation, used for the cyclic tests is shown in Figure 3.5.

![Cyclic Testing Protocol](image)

Figure 3.5. Cyclic testing protocol used for quasi-static connection tests.

To avoid fatigue of the connectors observed and documented during numerous tests, the cycle sequences with reduced load after reaching certain deformation (2.5 times yield displacement) were abandoned. The adopted cyclic protocol was a constant frequency protocol with one full cycle being completed in ten seconds. The calculation of the important properties based on the test data was done according to the proposed European standard (CEN, 1995).
3.2.4 Material Properties

At the end of each test, a small wood specimen was cut from each brace in the vicinity of the connection, so that the moisture content and density of the specimens could be determined. The average moisture content of the wood was found to be 12.6 % (with standard deviation of 3.5 %), while the average wood density was 403.8 kg/m³ with standard deviation of 3.8 % (Figure 3.6). The glulam brace members used in quasi-static tests were cut from larger glulam beams, 130 by 304 mm in cross section and 4.88 m (16-feet) long. These beams were used in a previous research project concerned with strength properties and failure modes of glued-laminated beams in bending (Timusk, 1996). The average modulus of elasticity (MOE) for these 29 beams, determined using standard ASTM three-point load test procedure was found to be 9,936 MPa with a standard deviation of 4.7 %. Since the modulus of elasticity as a material property remains relatively unaffected by the size of the wood member, the values above were considered valid for this study as well. The MOE for each of the beams and the cumulative distribution curve for the MOE are presented in figure 3.7.

![Relative Density of Glulam Connection Specimens](image)

Figure 3.6. Relative wood densities of the wood specimens.
Experimental tests were also conducted to determine the load-deformation relationships of the 12.7 mm diameter bolts used for brace connection tests. The tests were conducted by students enrolled in the Civil Engineering course CIVIL 321 – ‘Laboratory Project in Engineering Materials’. The monotonic tension tests performed on four replicates of 12.7 mm bolts showed an average ultimate tension stress of 497 MPa. The load-deformation curves for the four bolt replicates are shown in Figure 3.8. The lack of yielding plateau in the load-deformation behaviour indicates a material that has been cold worked which results in reduced ductility capacity. The bolts did, however, demonstrate strain hardening and necking which characterises a plastic behaviour with some ductility prior to failure. One of the load-deformation curves in Figure 3.8 showed a deformation offset due to slipping in the grip mechanism.
3.3 RESULTS AND DISCUSSION

3.3.1 Monotonic Tension Tests

During the monotonic tension tests, the glulam riveted connections yielded in a ductile single shear mode, while the bolted connections yielded in a double shear mode as expected. It was found, however, that yield modes and failure modes for the same connection could vary considerably. For glulam riveted connections the early behaviour was almost completely governed by yielding of the fastener, while the failure mode was a partial fastener pullout from the wood. In the case of bolted connections, a four plastic hinge double-shear-yielding mode was noticed with extensive wood crushing, but the failure mode was always splitting of the main wood member. Splitting of the wood always occurred at the maximum load. However, most of the connections were able to carry a significant percentage of the maximum load for some time after the splitting had occurred. Load-deformation curves obtained from monotonic tension tests for all connections from the first test group are presented in Figure 3.9.
Properties such as initial stiffness, ultimate load, yield load, ultimate displacement and ductility were determined from the tests and are presented in Table 3.2. One of the most important parameters determined was the yield deformation $\Delta_y$, because the cyclic testing protocol was defined in terms of $\Delta_y$ for each connection. For calculating the properties given in Table 3.2, the procedure described in the European CEN protocol was used (CEN, 1995). According to the procedure, initial stiffness of the connection is defined by the line that connects two points on the load-deformation curve at 0.1 $F_{\text{max}}$ and 0.4 $F_{\text{max}}$ respectively. The yield deformation is then defined as the deformation at the interception of the initial stiffness line and a tangent line with stiffness equal of 1/6 of that of the initial one. The yield load was the load on the curve that corresponds to the yield deformation, while the ultimate deformation was determined as deformation at which the load drops at 80% level of the maximum load. The overstrength factor was calculated as a ratio of the maximum load versus the design load according to CSA086.

Table 3.2. Average connection properties obtained from monotonic tension tests.

<table>
<thead>
<tr>
<th>Fastener</th>
<th>Bolts</th>
<th>Rivets</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter (mm)</td>
<td>9.5</td>
<td>12.7</td>
</tr>
<tr>
<td>Test group</td>
<td>I</td>
<td>II</td>
</tr>
<tr>
<td>Number of Specimens</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Yield load $F_y$ (kN)</td>
<td>133.9</td>
<td>43.7</td>
</tr>
<tr>
<td>Yield displacement $\Delta_y$ (mm)</td>
<td>2.35</td>
<td>1.37</td>
</tr>
<tr>
<td>Maximum load $F_{\text{max}}$ (kN)</td>
<td>163.7</td>
<td>64.3</td>
</tr>
<tr>
<td>Displacement at $F_{\text{max}}$ (mm)</td>
<td>3.91</td>
<td>4.73</td>
</tr>
<tr>
<td>Ultimate deformation $\Delta_u$ (mm)</td>
<td>7.84</td>
<td>6.58</td>
</tr>
<tr>
<td>Initial stiffness (kN/mm)</td>
<td>62.0</td>
<td>36.5</td>
</tr>
<tr>
<td>Ductility ($\Delta_u / \Delta_y$)</td>
<td>3.33</td>
<td>4.8</td>
</tr>
<tr>
<td>Overstrength ($F_{\text{max}} / F_{\text{design}}$)</td>
<td>2.54</td>
<td>2.65</td>
</tr>
</tbody>
</table>
Figure 3.9. Load-deformation curves from monotonic tension tests of the first test group.
3.3.2 Cyclic Tests

3.3.2.1 Load-Deformation Behaviour

The force-deformation hysteresis curves obtained during the cyclic tests were the most important set of data needed to assess the seismic performance of the connections. Typical load-deformation relationships of the connections from the first test group are shown in Figure 3.10 from a) to d) while the results from the second test group are shown in Figure 3.11. a) to c). The load value in the figures is the measurement obtained from the load cell, while the deformation is the average value obtained from both DCDT connection slip measurements.

![Hysteretic curves from cyclic tests of connections from the first test group.](image)

Figure 3.10. a) to b) Hysteretic curves from cyclic tests of connections from the first test group.
Figure 3.10. c) to d) Hysteretic curves from cyclic tests of connections from the first test group.

As evident from the results of both test groups, significant pinching of the hysteresis curves occurred. This is a very common feature for connections in timber structures and is a result of the irrecoverable crushing of the wood that leaves a gap at load reversals. During subsequent excursions through this gap region, lateral resistance and energy dissipation almost entirely occurs in the metal connectors. The first loop in a cycle of three therefore is the widest and shows the highest resistance, while subsequent cycles are narrower and typically achieve lower resistance for a given displacement. Previous research results suggested that this degradation of strength stabilises after three cycles and the third cycle is therefore often considered to represent the actual resistance when repeated (cyclic) loading is expected, such as for an earthquake. Figure 3.12. shows some of the stabilised (third cycle) loops at different deformation levels for a
bolted and riveted connection. The initial cycle, which follows the envelope curve, is thus not shown.

Figure 3.11. a) to c) Hysteretic curves from cyclic tests of the connections from the test group II.
It was found that in bolted connections the pinching effect was most significant at low deformation levels, while the hysteresis loops were getting thicker as the deformation increased. This is due to a change in deformation mode, from pure wood crushing at small displacements to bolt bending at larger displacements. The opposite was found to hold for the riveted connections, where rivet bending started from the beginning. Since the area inside the hysteresis loop for each cycle represents the amount of energy dissipated during that cycle, pinching in timber braced frames significantly reduces the hysteretic damping of the structure. Recent test results have shown, however, that the shape of the hysteresis loop (pinching) is not the single most important parameter for adequate seismic behaviour of timber structures (Buchanan 1988). The ability of the structure (connection) to sustain large deformations without significant deterioration in strength, is also very significant.

Figure 3.12. Stabilised hysteresis loops of bolted and riveted connection at different deformation levels.
3.3.2.2 Connection Properties

The locus of the extremities of the hysteresis curves is called the backbone or first cycle envelope curve, while the locus of the third cycle loops is called the stabilised (third cycle) envelope curve. Various measures of strength, stiffness and ductility can be obtained from the envelope curves. A summary of some of the average connection properties obtained from cyclic tests is given in Table 3.3 and Table 3.4. All properties in Table 3.3. are based on non-stabilised (first cycle) envelope curves, while properties in Table 3.4. were calculated using the stabilised envelope. All properties were again determined using the CEN Standard.

Table 3.3. Average connection properties from cyclic tests based on first-cycle envelope curve.

<table>
<thead>
<tr>
<th>Fastener</th>
<th>Bolts</th>
<th>Rivets</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter (mm)</td>
<td>9.5</td>
<td>12.7</td>
</tr>
<tr>
<td>Test group</td>
<td>I</td>
<td>II</td>
</tr>
<tr>
<td>Yield load $F_y$ (kN)</td>
<td>136.7</td>
<td>40.5</td>
</tr>
<tr>
<td>Yield displacement $\Delta_y$ (mm)</td>
<td>2.07</td>
<td>1.90</td>
</tr>
<tr>
<td>Maximum load $F_{\text{max}}$ (kN)</td>
<td>164.5</td>
<td>54.2</td>
</tr>
<tr>
<td>Displacement at $F_{\text{max}}$ (mm)</td>
<td>3.50</td>
<td>3.85</td>
</tr>
<tr>
<td>Ultimate deformation $\Delta_u$ (mm)</td>
<td>5.53</td>
<td>5.18</td>
</tr>
<tr>
<td>Initial stiffness (kN/mm)</td>
<td>72.8</td>
<td>27.5</td>
</tr>
<tr>
<td>Ductility ($\Delta_u / \Delta_y$)</td>
<td>2.67</td>
<td>2.72</td>
</tr>
<tr>
<td>Overstrength ($F_{\text{max}} / F_{\text{design}}$)</td>
<td>2.55</td>
<td>2.23</td>
</tr>
</tbody>
</table>
Table 3.4. Average connection properties from cyclic tests based on stabilised envelope curve.

<table>
<thead>
<tr>
<th>Fastener</th>
<th>Bolts</th>
<th>Rivets</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter (mm)</td>
<td>9.5</td>
<td>12.7</td>
</tr>
<tr>
<td>Test group</td>
<td>I</td>
<td>II</td>
</tr>
<tr>
<td>Yield load $F_y$ (kN)</td>
<td>128.7</td>
<td>37.4</td>
</tr>
<tr>
<td>Yield displacement $\Delta_y$ (mm)</td>
<td>2.08</td>
<td>1.95</td>
</tr>
<tr>
<td>Maximum load $F_{\text{max}}$ (kN)</td>
<td>152.8</td>
<td>47.9</td>
</tr>
<tr>
<td>Displacement at $F_{\text{max}}$ (mm)</td>
<td>3.68</td>
<td>3.68</td>
</tr>
<tr>
<td>Ultimate deformation $\Delta_u$ (mm)</td>
<td>5.35</td>
<td>4.96</td>
</tr>
<tr>
<td>Initial stiffness (kN/mm)</td>
<td>67.2</td>
<td>23.9</td>
</tr>
<tr>
<td>Ductility ($\Delta_u / \Delta_y$)</td>
<td>2.57</td>
<td>2.54</td>
</tr>
<tr>
<td>Overstrength ($F_{\text{max}} / F_{\text{design}}$)</td>
<td>2.36</td>
<td>1.97</td>
</tr>
</tbody>
</table>

Although the relatively small number of replicates tested prevents any statistically supported conclusions to be drawn, some definite trends could be observed. The obtained average ultimate load from non-stabilised curves was from 1.7 times (riveted connections) to 2.8 times (12 mm bolts) the design load. Connections with larger diameter bolts showed a smaller overstrength factor, defined as a ratio of average ultimate load versus design load. Riveted connections in general, showed a lower overstrength factor than connections with smaller diameter bolts, but they showed a higher ductility and lower variability between the test specimens. An interesting finding was that connections with two rows of bolts (group I) showed a higher overstrength factor than the connections with one row of bolts (group II). This may happen due to a number of different factors, one of them being the existing row modification factor (0.8 for two rows) in the Canadian design equations.
Some differences were noticed in properties obtained from quasi-static monotonic and cyclic tests of the same connections. In general, average curves obtained from monotonic tests showed higher values for maximum load and ductility than the average non-stabilised (first cycle) envelope curves (Figure 3.13). Stabilised (third cycle) envelope curves also showed lower ductilities than the corresponding first cycle backbone curves. In addition, the deformation at maximum load obtained during the monotonic tests was larger than the corresponding one obtained during the cyclic tests for almost all connections. These findings indicated that a connection subjected to cyclic loading attains its maximum load at a lower deformation than if
subjected to monotonic loading. In general, the monotonic tension tests overestimated the load-deformation behaviour of a connection subjected to cyclic loading. Although a small number of specimens was tested and the obtained hysteresis curves depended on the testing protocol (rate of displacement, number of cycles with high amplitude etc.) results from monotonic tension tests should be used with caution when determining the seismic properties of a timber connection or component.

The average capacity per rivet for the riveted connections of the first test group (3.2 kN/rivet) was lower than the corresponding capacity (3.5 kN/rivet) of the connections of the second test group. This is a very interesting finding which suggest that there might be a certain "size effect" or "group effect" in riveted connections indicating that larger connections have a lower capacity per rivet than the smaller ones. Further studies, however, are needed before any statistically supported conclusions can be made.

3.3.2.3 Energy Dissipation

For the cyclic tests, the area enclosed by each loop provides information on the amount of the hysteretic energy dissipated by the connection. For any cycle at a given deformation level, the largest amount of energy dissipation occurred the first time that a deformation level was reached, for all connections. The area enclosed by the first loop represents mostly the energy absorbed by the wood, which typically suffered unrecoverable deformation in the regions surrounding the fasteners. Any subsequent slip to the same deformation level resulted in a hysteresis loop that showed less energy absorption. The energy absorption further decreased during the third cycle at the same slip level, but the decrease was of much smaller magnitude than the decrease between the first and the second cycle.
During the cyclic tests, different rates and amounts of energy absorption (dissipation) were experienced for different connections. The dissipated energies for all specimens tested from the first test group are shown in Figure 3.14. The riveted connections showed the largest amount of energy dissipated. In addition, the energy dissipation curves for the three riveted specimens are very close together suggesting consistent energy dissipating characteristics. Bolted connections with smaller diameter bolts showed smaller and not so consistent dissipating characteristics as the riveted connections, but still much better than the connections with 19 mm bolts. These connections dissipated a very small amount of energy during the cyclic tests, as a result of their almost brittle behaviour.
3.3.2.4 Failure Modes

During the alternating load cycles, bolted connections yielded in a double shear mode with extensive wood crushing in the holes near the surface areas. Four plastic hinges were created on each bolt in the process. Glulam riveted connections yielded in a ductile single shear mode with one plastic hinge, typical for a nailed type connector, in both tension and compression half cycles. Extensive wood crushing was observed on both sides of the rivet shanks as well.

Similarly to the monotonic tests, yield modes and failure modes observed for same connections were different. At the end of the tests, glulam riveted connections failed with almost complete fastener pullout from the wood. (Figure 3.15). Bolted connections always failed in tension, with every failure being marked by a clearly audible release of energy and the immediate formation of a large crack or split. In bolted connections with two rows of bolts, the initial splitting always occurred along one row of bolts. In the next few cycles that process was followed by either increased splitting along the initial crack, or by splitting along the other row of bolts.

Figure 3.15. Typical riveted connection failure mode.
Splitting always occurred at maximum load, after which a drop in the load carrying capacity was noticed. The bolted connections with slender bolts (9.5 mm or 12.7 mm) were able to carry a significant portion of the maximum load for few more cycles after the splitting had occurred, thus showing a relatively ductile behaviour, which is so desirable in seismic design. Bolted connections with 19 mm bolted connections showed a steep drop in the connection resistance after reaching the maximum load. Riveted connections, on the other hand, showed very ductile behaviour and were able to carry a significant portion of the load even at high deformation levels. An example of a cracked wood member used for a 12.7 mm bolted connection after a cyclic test is shown in figure 3.16.

Figure 3.16. A glulam member used for bolted connection after cyclic testing a) cross section; b) side view.
3.4 SUMMARY

As evident from all the results presented, glulam riveted connections showed superior performance in all parameters compared to the bolted connections tested for similar design load levels. Furthermore, they exhibited a capability of resisting many load reversals without significant strength deterioration. Under reversed cyclic loading, riveted connections developed some slackness and finally complete pullout, which resulted in severely pinched hysteresis loops. Overall, large displacements were typically possible before failure, which permits ample warning before structural failure.

In bolted connections with lower bolt slenderness ratios (19 mm bolts), high flexural rigidity of the bolts resulted in more uniformly distributed wood bearing stresses along the bolt, which precipitated abrupt wood splitting and sudden loss of bearing capacity. Connections with bolts of higher slenderness ratios (smaller diameter bolts) exhibited more desirable behaviour in a sense that more wood crushing could occur before fracture, although eventually wood splitting was consistently the failure mechanism for all bolted connections tested.
4. SINGLE BRACE SHAKE TABLE TESTS

Understanding what a structure or structural component experiences in a real earthquake is a very complex matter. Shake table tests are increasingly being used to evaluate the seismic performances of structures or structural components and have many advantages over other testing methods. Since the shake table is driven in real time to follow an acceleration record, the forces and displacements are generated by the dynamic response of the structure. Because the rate-of-loading effects are solved for full-scale models, shake table testing is a valuable tool for these types of studies. The distributed inertia forces can be modelled correctly and the failure modes associated with a 3-D response can be entirely reproduced. Disadvantages include the specimen size limitations due to size of the shake table or hydraulic flow capacities, relatively more expensive and complex tests to conduct and operate, and finally, the results are strictly applicable only to the input ground motion used in the test.

4.1 OBJECTIVES AND SCOPE

A series of shake table tests on a full-scale single braced frame with two connections was conducted at the Earthquake Engineering and Structural Dynamics Research Laboratory (Earthquake Laboratory) at the Department of Civil Engineering, University of British Columbia. The tests were the second phase of the experimental program on the seismic
Single Brace Shake Table Tests

behaviour of braced timber frames. The objectives of the single brace shake table tests were the following:

• to determine the influence of the dynamic rate of loading on the load deformation behaviour of different connections;
• to compare the connection properties obtained from shake table tests to those obtained from quasi-static tests;
• to provide experimental data to calibrate the analytical connection models developed on the basis of quasi-static tests;
• to assess the ability of the four-hinged steel frame setup to adequately represent the behaviour of a single storey braced timber frame;
• to assess the feasibility of using UBC’s Earthquake Laboratory to conduct these types of tests.

4.2 DESCRIPTION OF THE TESTING FACILITY

The Earthquake Laboratory at UBC is 16.4 m long and 11.5 m wide, providing a space for construction, assembly and handling of relatively large structural models. The laboratory is equipped with an advanced, closed loop, servo-controlled hydraulic seismic simulator (shake table). The shake table is a 40 cm thick aluminum cellar platform, 3m by 3m in size, with a total weight of 20.5 kN and a total payload capacity of 156 kN. The table has a grid of 38 mm diameter holes that are used to attach the test specimens or equipment. The laboratory has a 4.2 m clearance over the table and is equipped with a 44.5 kN overhead crane for placing models and equipment. The aluminum platform and attached hardware were designed to have a
fundamental frequency of about 40 Hz, so it can be considered rigid within the operating frequency range of the shake table tests, which are mostly between 1 and 25 Hz.

The shake table can be configured to produce two types of multi-directional motions. The first configuration (called 1H-3V; 1 horizontal and 3 vertical) can be used to simulate longitudinal, vertical, pitch and roll motions. The other configuration (called 3H) can be used to simulate longitudinal, lateral and torsional (yaw) motions. In the 3H configuration, the horizontal longitudinal motions are produced by one hydraulic actuator with a maximum peak to peak displacement of 15.2 mm (6 inches). The displacement of the table is limited by the stroke of the actuators of ± 7.6 mm. The flow rate in the servo-valves limits the maximum velocities in both directions to 100 cm/sec. The maximum acceleration is limited by the force limits of the actuators (156 kN for the longitudinal, and 90 kN for the transverse) together with the table-specimen system. The actuator force reactions are resisted by a massive reinforced concrete foundation, extending around the table in the form of open box with a thickness of 1.5 m on the sides where the actuators are installed.

The shake table is controlled by a signal processing subsystem, driven by replication multi-shaker control software. This software performs a closed loop control of the shake table, which is capable of replicating recorded earthquake motions with high accuracy. The high performance digital control system can easily replicate earthquake motions for models with different mass-stiffness characteristics, which is a desirable feature for comparative studies on different models under similar loading conditions. The data acquisition system in the laboratory can record up to 128 channels of data from a test specimen. From this number, a total of up to 44 channels can be
conditioned by the variable gain buffers and cut-off filters, which provide optimal control over signal levels and noise reduction accurate for retrieval of dynamic testing data.

4.3 SHAKE TABLE TEST SETUP

A steel frame built for performing dynamic tests on timber shear walls and other components was used as the basic test setup. A simplified 2-D schema of the test setup is presented in Figure 4.1. The four-hinged steel frame was designed to provide vertical support for an inertial mass in form of three concrete blocks, placed on the top of the frame. Because of four hinges in one plane, the frame has no lateral resistance, so the only member that provides lateral resistance to the generated inertial forces is the component tested. The tested component in this case was the diagonal wooden brace with connections on both ends.

Figure 4.1. Simplified scheme of the test setup for the shake table tests of single braced frames.
The testing frame was actually a 3-D structure, consisting of two planar four hinged frames, placed parallel to each other approximately 1.2 m apart. A photo of the frame with the wood brace member inserted and ready for testing is shown in Figure 4.2. More detailed sketches of the front view and side view of the frame are given in Figure 4.3 and Figure 4.4, respectively.

Figure 4.2. Single brace shake table setup.

As presented in the figures, each of the frames was mounted on a large welded wide flange steel beam, which was then bolted down to the shake table. One smaller welded steel beam was placed in the middle of the testing frame, halfway between the two larger beams and was bolted to the shake table as well. The two planar frames were connected with cross beams (Figure 4.4) as well as with diagonal steel cross braces to improve the out-of-plane stiffness of the frame.
Three channel section beams were formed the top platform of the frame and provided fixture capabilities and vertical support to the three concrete blocks representing the inertial mass. The three concrete blocks had a total mass of 4,500 kg (10,000 lbs.). Because the frame has no lateral resistance of its own, a cantilever type steel stopper near the top of each column is provided to prevent excessive lateral movement or collapse of the frame after a failure had occurred in the tested component. Rubber rollers were mounted on the crane rail above the frame. They were in light contact with the middle concrete block to ensure pure in-plane motion of the frame and limit out-of-frame deformations.
Several modifications to the original design of the frame were made to enable it to accommodate wood brace members. One of the crossbeams of the frame near the top (Fig. 4.4) was modified to accommodate the diagonal brace element intended for testing. A picture of this detail, referred to as detail A in Figure 4.2, is given in Figure 4.5 a). The wooden brace element was held at the bottom by a specially designed fixture (detail B) that was attached to the specimen attachment beam (Figure 4.4 and 4.5 b). A load cell was attached at the top end of the brace to directly measure the axial load induced in the member. Rotational pins were introduced on both ends of the brace to ensure a pure axial load state in the brace and to prevent bending stresses from affecting the load cell readings. Because of the weight of the setup at the top of the
diagonal brace (fixture plus load cell), vertical motion was expected to be generated by the shaking. For that reason, rollers were attached to the top connection to avoid out of plane buckling of the brace and ensure that the connection slip develops along the brace (Figure 4.5 a).

Figure 4.5. Connecting the wood brace member to a) top and b) bottom of the testing frame.

Although the tested structure consisted of two very different materials, the whole test frame could be considered to resemble a single storey timber braced frame for two reasons. Firstly, braced timber frames are generally designed to resist the lateral loads only, while the vertical loads are resisted by some other system. In our case, the steel frame resists the vertical loads while the brace will take only the lateral loads induced by the inertial mass. Secondly, in seismic design of braced timber frames only the braces are expected to exhibit a non-linear behaviour, while all other members are typically designed to remain in the linear elastic range. That is exactly the performance expected from the test frame. Regardless of the material of all the other elements, the non-linear behaviour of the brace connections are expected to govern the system response.
4.4 SPECIMEN CONFIGURATION

The diagonal braces used in the tests consisted of SPF glued laminated timber, 130 x 152 mm (5" x 6") in cross section. They were exactly the same as the members used for quasi-static tests. Brace specimens were approximately 3.1 m long and had the same connection fabricated on both ends prior to testing. A total of eight braces (frames) were tested utilising five different connections, four of them previously tested quasi-statically. Most connections from the first test group of the quasi-static tests experienced relatively high maximum loads that could not be generated on the shake table due to various limitations. For that reason, only braces with connections from the second test group were tested, plus the 19 mm bolted connection from the first test group. For these connections the configuration was exactly the same as in the quasi-static tests, so all details can be found in Table 3.1 of the previous chapter. The connectors used were of the same material as those used in the quasi-static tests namely ASTM A307 bolts and 65 mm long, high-strength glulam rivets. Thickness of the steel plates was again 12.7 mm for the bolted connections and 6.4 mm for the riveted connection.

Two replicates were tested from each brace with different connections, except for the 9.5 mm bolted connection, which had only one replicate tested. In order to investigate the effectiveness of connection reinforcement, one of the brace specimens with 9.5 mm bolted connections was reinforced with steel threaded rods. Two threaded rods 6.4 mm in diameter (1/4 inch) were placed halfway between the last bolt and the end of the brace member. The rods were hand driven into pre-drilled undersized holes in the wood, placed perpendicular to the line of the bolts. Although testing only one specimen is far from being satisfactory from a statistical point
of view, especially for a material such as wood, the objective was to notice any difference in behaviour with significance beyond the statistical variability.

All glued laminated members were conditioned for five months prior to testing in a laboratory environment at an average temperature of $20 \pm 3^\circ C$ and relative humidity of $50\% \pm 10\%$. This was deemed to be a sufficient period of time for members to reach the equilibrium moisture content in dry service conditions. The bolted connections were fabricated a maximum of four days prior to testing, while riveted connections were conditioned for an additional three weeks to allow for relaxation of the wood fibres around the rivets.

4.5 INSTRUMENTATION AND DATA ACQUISITION

A number of instruments (sensors) were installed to monitor the dynamic behavior of the testing frame and the brace specimens. A total of ten different time history signals (channels) of digital information were recorded during each test. The location of the instruments is shown in Figure 4.6 and they are listed in Table 4.1. The axial load in the diagonal brace was measured directly from the in-line load cell at the top end of the brace (Ch. 8) with a maximum capacity of 222 kN (50,000 lbs.). The connection slip, as a difference of the deformation between the wood member and the steel side plate on one side, was obtained from displacement transducers (DCDTs) mounted at both connections (Ch. 6 and 7). The displacement transducers were calibrated for a range of $\pm 25.4$ mm (1 inch) with an accuracy of 0.1%. Strong motion accelerometers were placed at two levels, one at the top of the frame (Ch. 5) and the other at the level where the wood brace connects to the steel frame (Ch. 4). Both accelerometers recorded accelerations in east-west direction and had a range of measurement of $\pm 5g$ with a precision of 0.1%.
To characterize the response of the testing frame at the brace level, one displacement transducer (Celesco PT 101 Position Transducer) was placed at the same level with the acceleration sensors (Ch. 2). In addition, two more displacement sensors were placed to measure the vertical displacements between the shake table and brace member. The first recorded the vertical displacement between the shake table and the steel fixture connecting the brace (Ch. 9), while the second measured the vertical deformation to the wood brace, just below the connection (Ch. 10). By comparing the two records, information on the amount of rotation in the connection could be obtained. Shake table horizontal displacement and acceleration were measured with the built-in shake table sensors, a Linear Variable Differential Transducer (LVDT at Ch. 1) and a Kistler 8304 K-Beam accelerometer (Ch. 3).
Table 4.1. List of the instruments used and their location.

<table>
<thead>
<tr>
<th>Location</th>
<th>Instrument</th>
<th>Measurement</th>
<th>Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Displacement Sensor</td>
<td>Shake Table Displacement</td>
<td>Horizontal</td>
</tr>
<tr>
<td>2</td>
<td>Displacement Sensor</td>
<td>Frame Displacement at Brace</td>
<td>Horizontal</td>
</tr>
<tr>
<td>3</td>
<td>Accelerometer</td>
<td>Shake Table Acceleration</td>
<td>Horizontal</td>
</tr>
<tr>
<td>4</td>
<td>Accelerometer</td>
<td>Frame Acceleration at Brace</td>
<td>Horizontal</td>
</tr>
<tr>
<td>5</td>
<td>Accelerometer</td>
<td>Frame Top Acceleration</td>
<td>Horizontal</td>
</tr>
<tr>
<td>6</td>
<td>DCDT</td>
<td>Deformation of Lower Connection</td>
<td>Diagonal</td>
</tr>
<tr>
<td>7</td>
<td>DCDT</td>
<td>Deformation of Upper Connection</td>
<td>Diagonal</td>
</tr>
<tr>
<td>8</td>
<td>Load Cell</td>
<td>Axial Load in the Brace</td>
<td>Diagonal</td>
</tr>
<tr>
<td>9</td>
<td>Displacement Sensor</td>
<td>Displacement of the Fixture</td>
<td>Vertical</td>
</tr>
<tr>
<td>10</td>
<td>Displacement Sensor</td>
<td>Displacement of the Brace</td>
<td>Vertical</td>
</tr>
</tbody>
</table>

The information was recorded using one of the computer controlled data acquisition systems in the laboratory (Labview), with a sampling rate of 200 data points per second. All recorded signals were filtered with a 30 Hz low-pass filter using a 3 pole Bessel type filter with a 60 dB/decade roll off. Following each test, some of the recorded channel outputs were viewed on a monitor to verify the data. This quick verification was essential in detection of some recording problems during the tests. Furthermore, a video camera was used to capture the response of the entire frame under the earthquake loading for each test.
4.6 TESTING PROCEDURES

4.6.1 Impact Hammer Tests

The objective of the impact hammer tests was to obtain the frequency corresponding to the fundamental mode of vibration of the single brace model. It was desirable to know if the predominant frequency of excitation for the shake table tests was close to natural frequencies of the models. In addition, these tests were conducted in order to generate data for calibration of the analytical model developed later in the study. The hammer tests were performed at the Earthquake Laboratory with the testing frame and a brace already mounted on the shake table and ready for testing. The tests were conducted for all models with different brace connections.

An instrumented sledgehammer (Dytran model 5803A) was used for the testing. The hammer impacts were applied horizontally at the middle of the mass mounted at the top of the frame. The data measured during the tests included signals from the hammer and three horizontal accelerations, namely at the top of the frame, at the brace level (upper third), and at the shake table level. Each test consisted of three blows with the hammer with a time lag of few seconds between the blows. Each record had a total duration of approximately 15 seconds and the signals were recorded with a sampling rate of 200 samples per second.

4.6.2 Choice of Excitation Record for Shake Table Tests

During the shake table tests, only motions in the E-W direction of the frame were applied to provide an excitation of the braced frame model in the principal direction of stiffness. As
mentioned before, shake table displacements were controlled by a real time computer controlled system to reflect the acceleration input data from the earthquake record. The record, which was chosen among several existing earthquake records, had to provide a sufficient excitation to produce a non-linear response of the brace connections. To achieve this, it was necessary to select an acceleration record with a main frequency of excitation close to the natural frequency of the model. Natural frequency of the models with different brace connections were determined from an elastic modal analysis performed using the computer program SAP 90, supported by the results from the impact hammer tests.

Figure 4.7. Joshua Tree acceleration record: a) time history b) PSA response spectrum.
The record of 1992 Landers California earthquake, recorded at the Joshua Tree Fire Station, E-W direction (CSMIP, 1992), was chosen as the most appropriate for all the tests. The acceleration time history of the record and its pseudo spectral acceleration (PSA) response spectrum for 5% damping is shown in Figure 4.7. As shown in the figure, the earthquake record has a couple of peaks in the spectrum around 0.25 sec, which was very close to the initial period of the frame determined from the modal analysis and impact hammer tests. Another desirable characteristic of this record is that its highest peak of response of around 0.7 sec can induce a large amount of seismic energy to a structure with prolonged period of vibration after the initial shaking had taken place.

To maximize the response of the models, the maximum acceleration of the earthquake record was factored to produce the maximum allowed displacement by the shake table of ± 76 mm (3 inches). This acceleration level was then referred to as the maximum or 100% level, and all shake table tests were then performed using a maximum input acceleration at some percentage of that level.

4.6.3 Testing Sequence

To investigate the response of the braced frame models at different ranges, tests with low, moderate and high amplitude excitations were performed. At the beginning, low intensity tests with a broad frequency content were needed to tune the computer program that controls the shake table motion and to determine the interaction of the specimen and the shake table. Low level tests were also needed to provide information regarding the behaviour of the models in their linear range and their initial dynamic characteristics. Moderate intensity tests were very
useful in studying the non-linear load deformation characteristics of the brace connections at low deformation levels. Finally, tests at highest acceleration level possible were conducted to investigate the non-linear behaviour of the braces at larger deformation levels.

Table 4.2. A summary of shake table tests performed.

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Specimen Number</th>
<th>Brace Connection Type</th>
<th>Maximum Acceleration Planned (g)</th>
<th>Maximum Acceleration Recorded (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tune up 1</td>
<td>1</td>
<td>9.5 mm bolts</td>
<td>0.02</td>
<td>0.024</td>
</tr>
<tr>
<td>Tune up 2</td>
<td>1</td>
<td>9.5 mm bolts</td>
<td>0.05</td>
<td>0.038</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>9.5 mm bolts</td>
<td>0.05</td>
<td>0.042</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>9.5 mm bolts</td>
<td>0.26</td>
<td>0.31</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>9.5 mm bolts</td>
<td>0.4</td>
<td>0.43</td>
</tr>
<tr>
<td>4</td>
<td>2</td>
<td>12.7 mm bolts</td>
<td>0.26</td>
<td>0.32</td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>12.7 mm bolts</td>
<td>0.4</td>
<td>0.49</td>
</tr>
<tr>
<td>6</td>
<td>3</td>
<td>12.7 mm bolts</td>
<td>0.4</td>
<td>0.49</td>
</tr>
<tr>
<td>7</td>
<td>4</td>
<td>9.5 mm bolts + rods</td>
<td>0.26</td>
<td>0.31</td>
</tr>
<tr>
<td>8</td>
<td>4</td>
<td>9.5 mm bolts + rods</td>
<td>0.4</td>
<td>0.49</td>
</tr>
<tr>
<td>9</td>
<td>4</td>
<td>9.5 mm bolts + rods</td>
<td>0.53</td>
<td>0.62</td>
</tr>
<tr>
<td>10</td>
<td>5</td>
<td>Rivets</td>
<td>0.43</td>
<td>0.53</td>
</tr>
<tr>
<td>11</td>
<td>6</td>
<td>19 mm bolts</td>
<td>0.32</td>
<td>0.37</td>
</tr>
<tr>
<td>12</td>
<td>6</td>
<td>19 mm bolts</td>
<td>0.43</td>
<td>0.50</td>
</tr>
<tr>
<td>13</td>
<td>7</td>
<td>Rivets</td>
<td>0.43</td>
<td>0.50</td>
</tr>
<tr>
<td>14</td>
<td>7</td>
<td>Rivets</td>
<td>0.53</td>
<td>0.57</td>
</tr>
<tr>
<td>15</td>
<td>8</td>
<td>19 mm bolts</td>
<td>0.53</td>
<td>0.63</td>
</tr>
</tbody>
</table>

A summary of some of the most important details about shake table tests performed is given in Table 4.2. As presented in the table, a total of 15 tests were performed on eight single brace specimens with five different connections. For most of the tests, the maximum recorded shake table acceleration was slightly higher than the intended maximum acceleration for those tests. Some of the specimens were tested through several different acceleration levels in order to study the non-linear deformation development, and post moderate earthquake response. The other specimens, usually the second specimen of the pair of two, were tested to the highest
acceleration right on the undamaged state. This was intended to allow for comparison between the response of the model and the analytical results, which are presented in the next chapter, and to study the non-linear deformations in connections without any previous wood crushing around the connectors.

The procedure of mounting the frame on the shake table and installing the new brace for each shake table test was as follows:

- Position and fix the base of the four-hinged, collapsible steel frame on the shake table;
- Erect the frame until it reaches a vertical position;
- Hold the frame in a vertical position with a steel tube member (arm) attached on the other side to a reaction column near the shake table. This arm was attached to the test frame at the same height as the top of the wood brace;
- Bring the wood brace, with already fabricated connections on both sides, with a crane and attach it to the steel frame using the specially designed fixture;
- Release the arm so the wood brace starts to act as lateral load resisting system;
- Conduct a shake table test at a certain acceleration level. After the test, the frame is usually left to be in a non-vertical position;
- Position the frame in a vertical position and attach the steel arm to hold it in that position;
- Replace the wood brace member with the next one and continue with the testing.

### 4.7 MATERIAL PROPERTIES

Prior to the shake table tests, the modulus of elasticity (MOE) for all glulam brace members was determined. The braces used were cut from SPF glulam beams with length of 3658 mm (12
feet). As mentioned before the glulam beams were 130 mm by 152 mm in cross section with a total of four laminae along the section height. The MOE was obtained using the standard ASTM three-point load procedure. The tests were conducted at Forintek Canada’s Wood Engineering Laboratory in Vancouver. According to the ASTM procedure the glulam members were loaded with two concentrated forces, each one located at one third of the span of the beam (Figure 4.8). Three different quantities were measured during the tests: the load applied from the actuator (channel 1), the stroke (channel 2) and the mid-span deflection of the beam (channel 3).

![Test setup for determining the MOE of the glulam beams.](image)

A total of ten glulam beams were tested under load control. The load was gradually increased starting from zero to a specified maximum level. The maximum load was defined as half of the load that produces maximum elastic response in the beam, based on calculations done with material properties characteristic for SPF glulam members. The modulus of elasticity was then obtained as a function of the slope of the load-deformation line obtained from the tests. Based on the test results the average MOE was found to be 10,844 MPa (1,572.8 x 10^3 psi) with a coefficient of variation of 4.1%.
After the shake table tests were conducted, a small wood specimen was cut from the vicinity of the connections, so that a moisture content and density of the specimens could be obtained. The average moisture content of the wood was found to be 15.2 % with a coefficient of variation of 2 % while the relative density was 0.405 with a coefficient of variation of 3.5 %. Since the bolts used for the connections were the same ones previously used for the quasi-static tests, no additional tests on their material properties were conducted.

4.8 RESULTS AND DISCUSSION

4.8.1 Impact Hammer Tests

The impact hammer tests were conducted at the beginning of each test when a brace with different connections was installed. The fundamental natural frequency of each model was determined from the peak of the spectrum obtained by performing a fast Fourier transform (FFT) on the signals recorded on the top of the model. The fundamental frequency was found to be 4 Hz (0.25 sec) for all specimens (Figure 4.9). In other words, the influence of the connection stiffness on the initial period of the system could not be determined with the hammer test technique. The impact hammer tests were also used to determine the damping of the system. This was determined from the free vibration decay of amplitudes of horizontal acceleration measured at the top of the model. The damping was found to be about 2.2 % of the critical for the first mode of vibration of all models.
Figure 4.9. Typical amplitude spectrum of the system response during the impact hammer test.

4.8.2 Shake Table Tests

4.8.2.1 Data Pre-Processing

The most important results from any shake table test are the time histories recorded from the different sensors placed on the model. During the data analysis from the single brace shake table tests, emphasis was put on the different connection behaviour and its influence on the in-plane response of the frame. Before the data recorded from the sensors can be used for quantification of the dynamic response, a pre-processing of the signals was performed. The pre-processing was performed using a spreadsheet developed for that purpose in the commercial mathematical software package Mathcad. Firstly, different operations were needed to transform the recorded voltages to acceleration, displacement or force measurements. Then the signals were windowed and averaged based on the values of the initial voltage (quantity) associated with the zero reading of a given sensor.
At this point, the signals still contained some unwanted noise that did not result from the response of the model. There were usually two noise components in the signal. A high frequency noise was a result of an imperfection of the data acquisition system or of the instrumentation network. The high frequency noise was eliminated by using Butterfly type four pole low pass filters, with a cut-off frequency of 15 Hz. The low frequency noise in the signals can be triggered by the sensors themselves if not properly balanced or if not adequately mounted. This noise was eliminated by using two-pole Butterfly type filters with a cut-off frequency of 0.2 Hz.

4.8.2.2 Time History Response Parameters

After the pre-processing of the data was completed, a detailed data analysis in time domain was performed. Time histories of the response parameters such as acceleration, displacement, deformation in connections or load in the brace were plotted and carefully analysed. Time history plots of the shake table acceleration, acceleration at the top of the model, relative frame displacement at the brace level and the load in the diagonal brace for tests number 6, 8, 9, 10 and 12 are given in Figures 4.10 to 4.14 respectively.

Figure 4.10a. Time histories of selected quantities obtained from test number 6 – 12.7 mm bolts.
Acceleration is the most important quantity measured during a shake table test. This parameter is important not only because of its significance in fundamental laws of dynamics, but also because it is the only quantity measured during the real earthquake. Based on the analysis of the acceleration histories recorded from the single brace models, some conclusions can be made. Firstly, the maximum acceleration levels recorded at the shake table were higher than the planned levels for all tests (Table 4.2), which is common during shake table testing. Secondly, the peak accelerations recorded at the shake table level were higher than the corresponding peaks at the top of the model for all tests (Figures 4.10 to 4.14). Although the damping can partially be associated with the friction between the rubber rollers and the concrete block, still this suggests that the glulam braces with their non-linear connection behaviour during the tests acted as damper for the frame model.
Figure 4.11. Time histories of selected quantities from test number 8 - 9.5 mm bolts + rods.

Figure 4.12. Time histories of selected quantities from test number 9 - 9.5 mm bolts + rods.
Figure 4.13. Time histories of selected quantities obtained from test number 10 – Rivets.

Figure 4.14. Time histories of selected quantities obtained from test number 12 – 19 mm bolts.
The peak acceleration reduction was highest for test number 9, which is the model with reinforced 9.5 mm bolted connections, while it was the lowest for tests number 11, 12 and 15 for brace with 19 mm bolted connections. Even from such a crude analysis, it can be noticed that the maximal amount of energy was dissipated in the brace with 9.5 mm bolts with reinforcement bars. During this test, large non-linear deformations of the bolts and the reinforcement rods were experienced during the first earthquake burst, which later resulted in a failure of the top connection during the second earthquake burst. At this point, just after the 35th second of the record, the test had to be stopped because of safety concerns for the equipment.

By comparing the shapes of the input accelerograms (at the shake table level) and the accelerograms recorded at the top, some differences could be noticed. Although the top accelerogram shapes consisted of two main bursts in synch with the input ones, a drop in acceleration level was recorded in between the bursts (between the 18th and 25th second). The drop was again more significant for brace models with connections where larger non-linear deformations had occurred. This phenomenon is thought to be mostly due to gaps formed around the fasteners in the brace connections as a result of the wood crushing experienced by the demand from the first acceleration burst. The gaps formed in the connections significantly reduce the stiffness of the frame for the following low level deformations (accelerations), and thus have a vibration isolation effect in acceleration transfer along the height of the frame. In low intensity shake table tests for braces with all connections or in tests on braces with connections experiencing a lower level of non-linear deformations, this phenomenon was not so apparent.
By analysing the records from vertical displacements at the top connection (channels 9 and 10), and from visual observation during the test, relatively significant vertical component of motion was noticed. The difference in this vertical motion caused slight rotation in the connection at the top of the brace. In addition, a vibration of the cross steel beam where the brace is connected was also observed as a result of the axial force generated in the brace.

4.8.2.3 Load-Deformation Relationships

From the axial load in the brace measured directly and deformations of the brace connections, hysteresis load-deformation curves were obtained for both brace connections. It was visually noticed during the tests and later confirmed from the data analysis, that the top and bottom brace connections experienced different deformation levels. Usually during the initial phase of the test, due to numerous factors, including the variability of the strength properties of the wood, one of the brace connections started to experience more initial deformations and wood crushing. This resulted in concentration of the deformation demand for that connection later in the test. Load-deformation relationships for both brace connections recorded from tests number 3, 6, 9, 10 and 12 are shown in Figure 4.15.

Due to the test setup configuration and relatively big weight of the load cell and the steel fixture attached at the top of the brace, the top brace connection experienced higher deformations in almost all cases. This connection will be referred to as “the weaker” connection in the remainder of the chapter, while the connection where less non-linear deformations occurred will be referred to as “the stronger” connection. Regardless of which connection in a brace is weaker or stronger, this finding was very important for understanding the seismic behaviour of braced
timber frames. It showed that the deformation capacity of the brace is not equal to the twice the capacity of one connection tested separately. This fact was later used when developing the analytical models for braced timber frames.

Figure 4.15a. Selected load deformation relationships for both brace connections.
Figure 4.15. Selected load deformation relationships for both brace connections.

The hysteresis curves of the weaker connections obtained from shake table tests were used for direct comparison with curves obtained from quasi-static tests for corresponding connections.
Similarly as in quasi-static tests it was found that in bolted connections the pinching effect was most significant at low deformation levels, while the hysteresis loops were getting thicker as the deformation increased. The opposite was found to hold for the riveted connections. Similarly as in cyclic tests, by studying the envelope curves of the hysteresis loops various quantities such as strength, stiffness and ductility were obtained. Some of the average connection properties obtained from the weaker connections during shake table tests are given in Table 4.3.

Results from Table 4.3 have to be taken just as orientation values for several reasons. Firstly, a very limited number (one or two) of replicates from each connection were tested, so no statistically supported conclusions can be drawn. Secondly, the values presented can be treated as earthquake record dependent, which means they are valid only for the Joshua Tree Earthquake record chosen. Different earthquake records will require different deformation demand pattern on the connections, which may (or may not) influence the values of the quantities presented. Thirdly, during the shake table tests it was not always possible to obtain the ultimate load and deformation in the connections, even though records with peak acceleration of up to 0.53g were used, which was the highest acceleration possible that the test frame could be safely used. Both braces with riveted connections, the brace with reinforced bolted connections and one brace with 12.7 mm bolted connections experienced visible failure during the tests, so only for these connections can be assumed that the maximum load and deformation have been reached. The envelope curves of the five different weaker connections during the shake table tests at the maximum base acceleration level are presented in Figure 4.16.
Table 4.3. Connection properties for weaker connections from shake table tests.

<table>
<thead>
<tr>
<th>Fastener</th>
<th>Bolts</th>
<th>Rivets</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter (mm)</td>
<td>9.5</td>
<td>12.7</td>
</tr>
<tr>
<td>Test group</td>
<td>II</td>
<td>II</td>
</tr>
<tr>
<td>Yield load $F_y$ (kN)</td>
<td>43.1</td>
<td>38.7</td>
</tr>
<tr>
<td>Yield displacement $\Delta_y$ (mm)</td>
<td>1.72</td>
<td>1.39</td>
</tr>
<tr>
<td>Maximum load $F_{\text{max}}$ (kN)*</td>
<td>61.5</td>
<td>60.6</td>
</tr>
<tr>
<td>Displacement at $F_{\text{max}}$ (mm)*</td>
<td>4.05</td>
<td>4.27</td>
</tr>
<tr>
<td>Ultimate deformation $\Delta_u$ (mm)</td>
<td>6.1</td>
<td>6.54</td>
</tr>
<tr>
<td>Initial stiffness (kN/mm)</td>
<td>30.0</td>
<td>33.0</td>
</tr>
<tr>
<td>Ductility ($\Delta_u / \Delta_y$)*</td>
<td>3.5</td>
<td>4.7</td>
</tr>
<tr>
<td>Over-strength ($F_{\text{max}} / F_{\text{design}}$)*</td>
<td>2.54</td>
<td>2.15</td>
</tr>
</tbody>
</table>

* For some bolted connections $F_{\text{max}}$ might not have been reached.

Shake Table Test Envelopes for Different Connections

Figure 4.16. Shake table tests envelopes for different connections.

Despite the drawbacks mentioned before some observations can still be made. The maximum load obtained from the shake table tests for 12.7 mm bolted connection and the riveted connection were about 12 to 13 % lower, and for the 19 mm bolted connection 38 % lower than
the corresponding values obtained from the average first cycle quasi-static envelope curves. Although this is within the expected coefficient of variation for connections in wood structures, the lower maximum load obtained from shake table tests is in contradiction with the commonly held belief that wood as a material, (and thus wood joints and structures in general) has enhanced strength under short term dynamic loading. The lower resistance experienced during shake table tests may partially be contributed by the loosening of the connections and impact effects during the shaking. In addition, a vertical component of motion in the brace noticed during the tests, might also have contributed. Only the 9.5 mm bolted connection showed about 13% higher maximum load when compared to the average quasi-static results. Nevertheless, this value was close to the strongest specimen from the quasi-static tests so that is within the expected variability for timber connections.

The dynamic rate of loading had an influence on other observed connection parameters as well. The initial stiffness obtained from shake table tests for bolted connections was 10% to 40% higher than the value obtained from first cycle hysteresis curves, while the opposite effect was observed for riveted connections. Bolted connections also showed higher ductility during shake table tests (2.5, 3.5 and 4.7) compared to cyclic tests (1.9, 2.7 and 3.1), but still lower when compared to the static tests (3.0, 4.1 and 4.8). The opposite was found for riveted connections, the ductility of only 8 compared to 18 from cyclic tests. Although the shake table setup and the earthquake chosen can influence the dynamic tests results, it is evident that the envelope curves from monotonic tension tests overestimated the load-deformation characteristics of the connections subjected to earthquake ground motion. The envelope curves from cyclic tests also overestimated the connection behaviour from shake table tests. The difference, however, was of lower magnitude when compared to the static envelope results.
4.8.2.4 Frequency Domain Analyses

The influence of the brace connection on the period of the system during shaking was also studied. For that purpose, Fourier amplitude spectrum (FAS) analyses were performed on a number of acceleration histories recorded on the structure during the tests. Fourier amplitude spectra of the acceleration records at the brace level (channel 2) of frames with all five different connections along with the spectrum from the input acceleration, are given in Figure 4.17. Since the input acceleration record during all the tests changes only its intensity but not the frequency content, the Fourier amplitude spectrum for the input motion remains the same for all tests. Because of that, the changes in the frequency content of the FAS of different records from the models are solely the result of changes in the model itself.

Figure 4.17a. Fourier amplitude spectra of frames with different connections.
Unlike analysis of the dynamic amplification factor (DAF), which is highly influenced by the damping capacity of the structure, analysis of the dynamic characteristics of a record in the frequency domain reveals data on the stiffness changes only. By comparing the FAS of the response to the input spectrum, an estimate can be made on the period of vibration of the system and its changes during the shake table tests. Another way to investigate the frequency content of the response, which was not implemented here, is to calculate the frequency response function (FRF). The FRF takes into account the frequency content of the response with respect to the content of the input motion. In either case, only the approximate values of the vibration periods can be determined, since all the frequency components are not adequately represented in the input motion.

As presented in Figure 4.17, the frame with glulam riveted connections showed increased amplification in the range from 2.1 to 2.4 Hz., which probably coincides with the early period of shaking. Another amplification is noticed between 0.8 and 1.4 Hz., which corresponds to the later part of the shaking when extensive nonlinear behaviour was experienced. The spectrum of the frame with 12.7 mm bolted connections shows amplifications in a broader range, between 1.8 and 2.7 Hz. No amplification is noticed on the spectrum left from the main peak, which is at
1.5 Hz. The situation is similar on the left side of the spectrum of the frame with 19 mm bolted connections. Because of the stiffer nature of the connections with larger bolts, the amplification in the spectrum is mostly on the right side of the spectrum, between 1.8 and 3.25 Hz. The spectrum in this case has its peak values at 2.53 and 2.65 Hz., indicating much stiffer behaviour than frames with other connections. The frame with 9.5 mm reinforced bolted connection showed amplification between 1.7 to 2.4 Hz with the peaks around 1.8 and 1.9 Hz.

Figure 4.18. Dynamic amplification factor in frequency domain for frame with 19 mm bolts.

Another way to look at the dynamic characteristics of the braced frame models during shaking is to calculate the dynamic amplification factor (DAF) in frequency domain. This can be calculated as a simple ratio of the Fourier amplitude spectrum of the acceleration at the top of the frame versus the Fourier spectrum of the input motion. An example of DAF in frequency domain is given in Figure 4.18. for the frame with 19 mm bolted connections (run 12). As shown in the figure, the dynamic amplification peaks at around 2.55 to 2.75 Hz., which corresponds to the natural period of vibration, obtained previously.
4.8.2.5 Energy Dissipation

Another important parameter obtained from the shake table tests was the energy dissipation in the connections. As mentioned before, during the shake table tests, more deformation and energy dissipation was experienced in one of the connections, usually the upper one, while the other one remained less excited. The average energy dissipation in the brace connections, calculated as the area enclosed by the hysteresis loops, when subjected to approximately the same level of earthquake ground motion, is given in Figure 4.19. As shown in the figure, braces with smaller diameter bolted connections dissipated more energy than braces with larger diameter bolts. This finding is in accordance with the results from cyclic tests, and again makes slender bolts more desirable for seismic design purposes in braced timber frames.

![Shake Table Tests - Average Dissipated Energies](image)

Figure 4.19. Energy dissipation in different brace connections during shake table tests.
Riveted connections again experienced the highest amount of energy dissipation of all non-reinforced connections. The 9.5 mm bolted connection with reinforcement showed the highest energy dissipation of all connections including the rivets. In this case, after splitting of the wood had occurred, the energy dissipation was enhanced by the large non-linear deformations in the treadered rods. It was also found that the total energy dissipation in each connection during shake table tests was lower than the corresponding total energy dissipated during cyclic tests (Figure 4.20). This finding suggests again that the cyclic testing protocol, although modified to lower the demand on connections tested, still had a high energy demand when compared to a powerful and high-demand earthquake such as Landers.

Figure 4.20. Dissipated energy in connections during cyclic (right side) and shake table tests (left side).
4.8.2.6 Failure Characteristics

Because of the different acceleration levels and different connection properties, not all connections experienced failure during the shake table tests. Failure was experienced in tests number 6, 9, 10, 13 and 14. The connections which experienced failure did so in a similar manner as the connections during the cyclic tests. Bolted connections experienced splitting of the wood along the bolt line, while riveted connections experienced partial pullout from the wood. An example of failure modes in typical bolted and riveted connection during the shake table tests is shown in Figure 4.21. Bolted connection with reinforced trenched rods exhibited a less brittle failure mode accompanied by large non-linear deformations in the rods.

Figure 4.21. Typical failure mode in a) bolted and b) riveted connection during shake table tests.
4.9 SUMMARY

In this chapter, results were presented from a series of shake table tests conducted on single storey braced frame models with different connections. The tests were conducted as a second phase of the experimental program on seismic performance of braced timber frames. A four hinged steel frame built for performing dynamic tests on timber components was used as a basic setup. Diagonal braces with five different connection types were connected to the steel frame as a lateral load resistant system to be tested. Four connections utilised bolts as a fastener while only one connection utilised glulam rivets. Four of the connections were the same as the connections previously tested under cyclic load, so an estimate could be given on the influence of dynamic rate of loading on the connection load-deformation characteristics. The fifth brace connection was a bolted connection with a pair of threaded steel rods as reinforcement, inserted manually halfway between the last bolt of the connection and the end of the brace.

The results showed that the steel frame could be used successfully to investigate the seismic response of a single storey braced timber frame model. The seismic response of the braced models was found to be highly influenced by the connection details. Braces with small diameter bolts, glulam rivets or reinforced bolted connections showed desirable seismic performance. They were able to dissipate the highest amount of the seismic energy generated by the earthquake motion. It was also observed that the dynamic rate of loading has an influence on load-deformation connection properties, when compared to corresponding properties obtained from quasi-static cyclic tests. The influence of connection behaviour on the period of vibration of the frames was also studied. In general, a valuable information was collected from the tests, which will be used later in the analytical part of the study.
5. ANALYTICAL PREDICTION OF SINGLE BRACE SHAKE TABLE TESTS

Analytical modelling is very important link between experimental work and overall effort of understanding of seismic performance of wood buildings. Analytical models can be used to predict structural behaviour under specific seismic loading and perform extensive parametric studies on their seismic performance. They can also be used in development and calibration of code requirement and seismic design procedures. Since analytical models are, after all, simplified representation of a real structure, the calibration is of utmost importance. To do this, analytical results are compared to experimental findings to fine-tune the model parameters. This chapter deals with the analytical modelling of single storey braced frame a next step to the full analysis of multi-storey frames.

5.1 OBJECTIVES

A series of shake table tests on single braced frame models was described in the previous chapter. Non-linear load-deformation relationships for braces with different connections were obtained from the tests. The main objective of the research described in this chapter was to analytically predict the non-linear brace behaviour obtained during the shake table tests. For that reason, an analytical model was chosen to predict the non-linear behaviour of each of the different connections (braces) tested. The connection (brace) model was then incorporated into a mathematical model simulating the entire testing frame with the wood braces with different
connections. The seismic behaviour of the braces with different connections obtained analytically was then compared to the behaviour previously obtained from the shake table tests. Furthermore, the seismic behaviour of the entire frame obtained analytically was then compared to the response from the shake table tests.

5.2 ANALYTICAL MODEL DEVELOPMENT

The dynamic behaviour of structures subjected to earthquake ground motion can be simulated with numerous mathematical models using computer packages such as ANSYS, ABAQUS, CANNY, SAP90, SAP2000, RUAUMOKO, ETABS and many others. Some of them can perform a linear dynamic analysis in two or three dimensions, while others can perform non-linear static, pushover, or non-linear dynamic analysis of planar or 3-D structural models. Most of these programs contain models that have been developed to simulate the response of reinforced concrete or steel elements, without paying much attention to the specific characteristics of timber structures.

An analytical model is defined by a set of assumptions idealizing the relevant aspects of the real system that we want to model. The aspects that are idealized in the models are usually the constitutive assumptions (linear elastic, non-linear elastic, non-linear inelastic etc.), geometric complexity (one dimensional, 2-D or 3-D domain models), or type of loading and response (static, cyclic, dynamic or random). For modeling of the behaviour of single brace shake table tests, two types of models were developed: a 3-D linear elastic model and a 2-D (planar) non-linear model of the frame with different brace connection characteristics.
5.2.1 3-D Linear Elastic Model

The 3-D linear elastic model was developed using the computer program SAP90 (Wilson and Habibullah, 1992). Among other features, the program can perform modal analysis, linear elastic static and time history dynamic analysis. The time history dynamic analysis is performed by integration of a chosen number of modal responses. The three dimensional linear elastic finite element model is presented in Figure 5.1. All elements were modelled as linear elastic beam type finite elements with their exact length, cross sectional dimensions and material properties. The cross sectional properties of the four hinged steel frame were given in Figure 4.3 in the previous chapter. The glulam brace was modelled according to the properties obtained from the material tests also presented in the previous chapter. Although not observable in the impact hammer tests, the influence of different brace connections on the brace stiffness in the elastic range was taken into account when determining the stiffness of the brace.

![Figure 5.1. A 3-D linear elastic model of single brace frames in SAP90.](image)
The masses were concentrated at the top four nodes of the model. The lumped mass values at the nodes included the mass of the concrete blocks as well as the mass of all steel members of the upper half of the frame. A global constraint option of the program was used to constrain the displacements of the top nodes, in order for the in-plane rigidity of the steel grid supporting the concrete mass blocks. The base supports of the frame allowed free rotation in the in-plane longitudinal direction, while all other rotations were constrained. The glulam brace element was allowed free rotations at both ends in the plane of the seismic loading. For the linear dynamic analyses, the damping for the model was chosen to be the same as the value determined from the impact hammer tests.

5.2.2 2-D Non-Linear Analytical Model

Several researchers have proposed non-linear hysteresis models for timber connections, nailed shear walls or timber structures in general. Most of them require full scale testing on connections or other structural components prior to the analysis in order to determine the load-deformation characteristics for that particular element under cyclic loading. Although this is suitable for doing analyses for research purposes, it is a major impediment to the typical designer who has no access to a testing facility. Moreover, only a few of the models are incorporated in commercial packages for the non-linear analysis of structures.

One of those few packages is a version of the well known computer program DRAIN-2DX (Prakash et al. 1993), which incorporates a subroutine specially developed to evaluate the hysteresis behaviour of a timber structural component. The subroutine was developed by Dr. Ario Ceccotti and his collaborators at the University of Florence in Italy (Ceccotti and Vignoli,
The model reproduces the path of a typical wood connection hysteresis loop, keeping track of the maximum previous deformation, and using different slopes to take into account the complex connection behaviour associated with wood crushing and steel yielding. The model (Figure 5.2) is defined by a total of nine parameters, including six stiffness parameters ($K_1$ to $K_6$), two deformation parameters ($U_1$ and $U_2$), and one force parameter ($P_0$). The envelope curve is defined with three slopes: the initial stiffness $K_1$, the tangent stiffness $K_2$ and the degrading stiffness $K_3$. The model also includes an unloading stiffness $K_5$, an inner stiffness $K_4$ which defines the pinched loops for the subsequent cycles, and a return stiffness $K_6$. The force associated with zero deformation is defined as $P_0$, while the deformations at the intersections of the lines defining the envelope curve are denoted as $U_1$ and $U_2$.

![Mathematical model for connection behaviour used in the analyses.](image)

Since the computer program DRAIN-2DX supports only a non-linear dynamic analysis of planar structures, a two dimensional model was developed to represent the 3-D setup for the single brace shake table tests (Figure 5.3). In the 2-D model, an effort was made to simulate the properties of the 3-D four hinged steel frame and the inserted glulam brace in two dimensions as
accurately as possible. The non-linearity in the model was essentially concentrated in the glulam brace element, while all steel elements were modelled to be linear elastic. The mass of the concrete blocks and the weight of the steel grid supporting the blocks was equally divided between nodes five and six. The steel element number five was modelled to be very rigid representing the rigid grid supporting the concrete blocks. Both ends of the element were allowed to have free rotations in the plane of the frame, representing the upper two hinges of the four hinged frame. The base supports of the frame allowed free in-plane rotation, representing the bottom two hinges of the collapsible frame.

![Diagram of shake table test setup](image)

Figure 5.3. A 2-D non-linear DRAIN-2DX model of the single brace shake table test setup.

The two brace connections could not be represented separately in the model due to numerical instability of the computer solution in compression. For that reason in element seven, the linear elastic behaviour of the glulam brace was combined with the non-linear behaviour of both connections into one non-linear spring element. This procedure was used for all four different connections tested, so a total of four different brace (frame) models were created. Stabilized
hysteresis loops from the quasi-static tests were used as a starting point for development of the brace models. Those loops were then modified according to the findings and observations from the shake table tests. As mentioned in the previous chapter, during the shake table tests it was noticed that one of the connections always experienced a higher level of deformation while the other remained less excited. To model the behaviour of both connections as one element, the envelope of the weaker connection was added to the envelope of the stronger connection to form one common envelope. Then the linear elastic properties of the wood brace were incorporated in that common envelope. The first three stiffness slopes $K_1^*$, $K_2^*$ and $K_3^*$ of the model were then determined to follow this envelope. Since no significant differences were noticed in the unloading, inner and return slopes ($K_4$, $K_5$ and $K_6$) of the weaker and stronger connections during the shake table tests and quasi-static tests, the same values were used for both connections. The values for these slopes $K_i^*$ were determined according to equation 5.1., based on the theory of serial elastic springs, with the initial connection slopes $K_i$ and the elastic stiffness of the wood $K_w$ as known input.

$$K_i^* = \frac{1}{2} \frac{1}{K_i} + \frac{1}{K_w}$$ (5.1)

The deformations $U_i^*$ were then determined at intersections of the $K_i^*$ slopes. The original value of $P_0$ was not influenced by these transformations and was kept the same as for the connection tests. The steel crossbeam in the shake table setup, to which the diagonal brace was attached, was a part of the frame that could not be directly represented in a 2-D model. A significant vibration of the crossbeam was noticed during the shake table tests that influenced the load-deformation characteristics of the brace and the frame in general. For that reason, the stiffness of the crossbeam was indirectly included in the 2-D model. A linear elastic model of the grid,
formed by the two vertical columns and the crossbeam, was created in SAP90. The grid was assumed to be pin supported at the end of each column. Elastic stiffness of the grid was obtained by a static analysis of the model, loading the crossbeam with a force at angle at which the brace was attached to the crossbeam. Finally, this stiffness was added as a serial spring to the already determined stiffness slopes $K_i$ of the model, as described earlier, to modify the non-linear model for the brace into its final version used in the analyses. This procedure was repeated to determine the non-linear load-deformation characteristics of the four different brace models with four different connections. The final values of the parameters used to define each of the four different braces in the analyses are shown in Table 5.1.

Table 5.1. Final parameters used to model different braces using the Florence model.

<table>
<thead>
<tr>
<th>Connect.</th>
<th>$U_1$ (mm)</th>
<th>$U_2$ (mm)</th>
<th>$K_1$ (kN/m)</th>
<th>$K_2$ (kN/m)</th>
<th>$K_3$ (kN/m)</th>
<th>$K_4$ (kN/m)</th>
<th>$K_5$ (kN/m)</th>
<th>$K_6$ (kN/m)</th>
<th>$P_0$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.7 mm</td>
<td>2.67</td>
<td>7.43</td>
<td>16570</td>
<td>4980</td>
<td>-1110</td>
<td>9500</td>
<td>19910</td>
<td>940</td>
<td>5.5</td>
</tr>
<tr>
<td>9.5 mm</td>
<td>4.46</td>
<td>7.82</td>
<td>11220</td>
<td>3830</td>
<td>-530</td>
<td>8490</td>
<td>26820</td>
<td>1250</td>
<td>5.4</td>
</tr>
<tr>
<td>Rivets</td>
<td>2.0</td>
<td>14.0</td>
<td>17600</td>
<td>1720</td>
<td>-1050</td>
<td>5610</td>
<td>18330</td>
<td>120</td>
<td>3.0</td>
</tr>
<tr>
<td>19 mm</td>
<td>2.76</td>
<td>4.13</td>
<td>23910</td>
<td>9360</td>
<td>-10900</td>
<td>28890</td>
<td>42630</td>
<td>2000</td>
<td>8.5</td>
</tr>
</tbody>
</table>

For the non-linear analyses, a viscous damping ratio of 3.5% of the critical damping was used. For large deformations, the hysteretic damping was automatically included in the analysis by the load-deformation curve for the brace, so the importance of the viscous damping was diminished. For small deformations, however, the skeleton model stays in the elastic range so the model cannot include the hysteretic damping found in reality. Therefore, an addition of sufficient viscous damping to overcome this was important. The viscous damping coefficient, included in the analysis, was calculated using the damping ratio of 3.5% and the natural period of the frame obtained from impact hammer tests.
5.3 RESULTS AND DISCUSSION

5.3.1 Modal Analysis

Modal analyses were performed using both, the three dimensional linear elastic model and the two dimensional non-linear model, in order to obtain the natural frequencies of the models and to compare these with the measured values. In addition, the modal analysis was one of the tools to calibrate the analytical models used. For both models, natural frequencies were determined for all four cases of different brace connections. In the linear model, the influence of different connections on the brace stiffness was modelled by using wood brace elements with different cross-sectional areas. The cross-sections were chosen so that the final axial stiffness of the brace was equal to the initial brace axial stiffness plus the initial stiffness of both connections. As explained earlier, in the non-linear frame model, the influence of the connections was already accounted for in the brace behaviour.

Figure 5.4. Fundamental mode shape obtained from the linear elastic 3-D model (SAP90).

The fundamental mode shape obtained from the modal analysis of the linear 3-D model is shown in Figure 5.4. A second mode, associated with buckling of the columns of the frame, was also obtained from the analysis. Its importance, however, is not significant because its high
natural frequency of about 39 Hz, and its low participation in the model response of only 0.7 %. The first mode participation was found to be 99.3 % of the total response of the model. Based on
this, as well as on the observations and analyses from the shake table test data, it was evident
that the model response closely resembled that of a single degree of freedom system. As shown
in Figure 5.4, the first mode shape effectively captures the deformation of the crossbeam where
the brace was attached. The corresponding fundamental frequencies for the model with different
brace connection are given in Table 5.2. The obtained frequencies differ for about 12 % based
on the brace connection, but all were very close to the measured value of 4.0 Hz.

Table 5.2. Fundamental frequencies of the models with different brace connections.

<table>
<thead>
<tr>
<th>Model</th>
<th>Connections</th>
<th>Frequency (Hz)</th>
<th>Period (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>9.5 mm Bolts</td>
<td>4.03</td>
<td>0.248</td>
</tr>
<tr>
<td></td>
<td>12.7 mm Bolts</td>
<td>4.08</td>
<td>0.245</td>
</tr>
<tr>
<td></td>
<td>19 mm Bolts</td>
<td>4.31</td>
<td>0.232</td>
</tr>
<tr>
<td></td>
<td>Rivets</td>
<td>3.97</td>
<td>0.252</td>
</tr>
<tr>
<td></td>
<td>9.5 mm Bolts + rods</td>
<td>3.87</td>
<td>0.258</td>
</tr>
<tr>
<td></td>
<td>9.5 mm Bolts</td>
<td>3.94</td>
<td>0.254</td>
</tr>
<tr>
<td></td>
<td>12.7 mm Bolts</td>
<td>4.00</td>
<td>0.250</td>
</tr>
<tr>
<td></td>
<td>19 mm Bolts</td>
<td>4.22</td>
<td>0.237</td>
</tr>
<tr>
<td></td>
<td>Rivets</td>
<td>3.89</td>
<td>0.257</td>
</tr>
<tr>
<td></td>
<td>9.5 mm Bolts + rods</td>
<td>3.80</td>
<td>0.263</td>
</tr>
</tbody>
</table>

Table 5.2 also shows the fundamental frequencies obtained from the modal analysis of the non-
linear 2-D model, while the first mode shape is given in Figure 5.5. The 2-D model initial
frequencies were about 30 % lower for all connections when compared to the 3-D model with
the same mass. To get closer to the measured frequency, the influence of the crossbeam in the
total load-deformation behaviour of the brace was reduced, so the final frequency values
presented in Table 5.2 were obtained. Similarly as the 3-D model, the 2-D model showed almost single degree of freedom behaviour with a first mode participation factor of 99.2 %.

![Figure 5.5. First natural mode of the analytical non-linear 2-D model (DRAIN-2DX).](image)

### 5.3.2 Time History Dynamic Analysis

Time history dynamic analyses were performed using both analytical models for all cases of different brace connections. To enable a comparison with the shake table test results, the actual shake table accelerations recorded in each test were used as an input ground motion in the analyses.

#### 5.3.2.1 Linear Analysis

A number of time history analyses were performed using the linear model in SAP90. This type of analysis did not include the nonlinear characteristics in the brace, therefore no comparison could be made with the experimental results at the brace level. In addition, comparison of the results at the frame level was also hindered by the influence of the friction of the rubber rollers on the top of the mass, although the magnitude of that influence was difficult to be determined. Some efforts were made to account for it by adding an additional element in the linear as well in
the non-linear model, but without much success. The linear model, however, well represented
the 3-D nature of the testing frame and results could be used for preliminary analysis. For
example, a linear analysis with this model was first used before the shake table tests were
conducted to determine the approximate force level in the brace. Based on these results, a
decision was made on the type of connections that would be used for the tests.

Figure 5.6. A comparison of an acceleration history measured and obtained from SAP90 model.

The linear analysis showed higher acceleration levels at the top of the model in all simulations,
with peak accelerations being from 1.7 to 3 times higher than those recorded in the tests. Figure
5.6 shows a comparison of the acceleration history measured at the top of the frame for test
number eight and the history obtained at the top of the linear SAP90 model. This discrepancy
resulted partially form the nature of the linear analysis and partially form the damping from
rollers that could not be modelled. Generally, the linear analytical model showed a higher
frequency response with lower deformations at the top and higher forces generated in the brace. This observation supports the present seismic design philosophy, which recognizes the ductility as a structural ability to dissipate energy and reduce the intensity of the seismic forces involved. The displacement time history from test number eight measured at the brace level compared to the corresponding displacement history obtained from the linear SAP90 model, are presented in Figure 5.7.

Figure 5.7. A comparison of relative displacements measured and obtained from SAP90 model.

5.3.2.2 Non-Linear Analysis

A series of non-linear analyses were performed using the DRAIN-2DX model. All the different non-linear models representing the four different connections were used in the analysis. To permit comparison with the relative acceleration records generated by the DRAIN model, the
shake table accelerations were subtracted from the accelerations measured at the top of the frame. A comparison of measured accelerations at the top for three different tests and the histories obtained analytically are presented in Figure 5.8.

Figure 5.8. A comparison of acceleration histories measured and obtained from DRAIN models.

As shown in the figure, the maximum accelerations obtained by the analytical model were about 30% to 100% higher than the measured values. Although these differences were much smaller
than those obtained from the linear model, they were still significant. The differences were smaller (around 30%) for tests at lower acceleration levels, while they were greater for tests at higher acceleration levels. The inability to model the steel cross-brace and the roller friction at the top of the frame were the main reasons for the experienced differences. The differences in deformations were found to be smaller in magnitude than the differences in acceleration.

Figure 5.9. Load–deformation relationships of braces with different connections compared to the corresponding shake table test envelope curves.

Much better agreement was obtained when comparing the load-deformation relationship of the braces obtained analytically and during the tests. Figure 5.9 shows the load–deformation hysteresis loops of braces with four different connections compared to the corresponding shake table test envelope curves. The envelope curves were obtained by addition of the load-
Analytical Prediction of Single Brace Shake Table Tests

deformation characteristics of the stronger and the weaker connection from the shake table tests, at low and high deformation levels, as well as the elastic properties of the brace. The results on analytical load-deformation characteristics were considered satisfactory for simulating tests at higher acceleration levels. Because the adjustment of the brace model parameters was done using the larger deformation range, the response of the model to lower acceleration levels overestimated the experimental response.

Finally, a comparison of the dissipated energy in the braces with four different connections during the shake table tests and from analytical simulation is shown in Figure 5.10. Relatively good agreement between the results was found, especially for the braces with riveted connections (Run 10 for example). For some of the brace connections, like the 19 mm bolted connection (Run 12), analytical models showed increased energy dissipation during the second half of the response. This was due to increased hysteretic damping generated at small deformation levels in the model after the maximum deformations have been reached.

In other cases the analytical model overestimated the energy dissipation during the whole length of the record for some tests like number eight for example (Figure 5.10). This was because the damage caused in the connections during one test affected the response of the frame to subsequent excitations. This DRAIN model always captured the behaviour of the connection (brace) from its virgin state, including the energy dissipation from wood crushing at lower deformation levels. Part of this energy dissipation in reality occurred during the tests that were conducted prior to that particular test (test seven prior to eight for example) because they used the same brace specimen. The program DRAIN-2DX has an option to partially include this
residual damage by simulating consecutive excitations within one analysis. This analysis option was used on several models but without much success.

Figure 5.10. A comparison of the dissipated energies for braces with four different connections.
5.4 SUMMARY

Analytical models to simulate the behaviour of single braced frames with different connections were presented in this chapter. Results were presented from simulations with two different models, a 3-D linear elastic model and a 2-D non-linear model. The linear model was developed in SAP90, a computer program that can perform a three dimensional linear dynamic analysis. A linear response of the model provided relatively good results for very small excitation levels, yet for larger excitations the model greatly overestimated the forces and underestimated the displacements.

A non-linear model was developed using the DRAIN-2DX computer program. The program can perform non-linear dynamic analyses in two dimensions only, but it contains a subroutine that can simulate the specific type of nonlinearity found in timber connections. To model the behaviour of these connections in the analyses, typical load-deformation curves obtained from shake table tests were used. The accelerations obtained at the top of the model were higher than the values obtained during shake table tests, which was expected since the effect of the friction of the rubber rollers was not simulated. The load-deformation response obtained in the brace was relatively close to the experimental behaviour. In addition, the energy dissipating characteristics were found to be very close to the experimental values, especially for the braces with riveted connections.
6. FORCE MODIFICATION FACTORS FOR BRACED TIMBER FRAMES

The force modification factors (R-factors) in building codes account for the capability of the structure to absorb energy within acceptable deformations and without failure, and thereby reduce the seismic design forces. The factor also takes into account the existence of alternate load paths and redundancy of the structural system. A building designed with a value of $R$ greater than 1.0, which is the calibrated value for elastic response, is presumed capable of undergoing inelastic deformations. Different R-factors are assigned to different types of structural systems reflecting their design, construction experience and seismic performance during past earthquakes. Those types of structures that performed well during past earthquakes are assigned a higher R-factor in the National Building Code of Canada (NBCC).

6.1 OBJECTIVES

Values of force modification factors in design codes are based mostly on empirical observations of the behaviour of different structural systems during past earthquakes. Often there is very little theoretical or experimental background for the numerical values given in the codes. In the field of timber structures, the amount of information on R-factors is even less, compared to other types of structures. Consequently, the definition of R-factors requires considerable individual judgement, resulting in vastly varying values in different codes. For that reason, the main objective of the research presented in this chapter was to validate the current force modification
factors for braced timber frames in the National Building Code of Canada. In addition, an attempt was made to evaluate the R-factor dependency on the shape of the load-deformation characteristics of the braced connections, the energy dissipation and ductility characteristics of the system as well as on the initial period of the structure.

6.2 DEVELOPMENT OF THE ANALYTICAL MODELS

Analytical models based on experimental test results are very important to gain understanding of the seismic performance of structures. Once calibrated, these models can be used to predict the structural behaviour under specific seismic loading or to perform parametric studies to determine the influence of different design parameters on the seismic response. Based on the results from the experimental research presented in the previous chapters, non-linear analytical models were developed to predict the seismic behaviour of typical braced frame structures with different brace connections.

Figure 6.1. Elevation of braced timber frame of a typical industrial type building.
Analytical models representing two different types of structures with braced timber frames as lateral load resistant system were chosen as basic models for the analyses. The structures were chosen to represent the most common cases where braced timber frames can be used as lateral load resistant systems. The first structure was a typical industrial building, such as a laboratory or warehouse (Figure 6.1). An example of such a system is the Wood Engineering Laboratory of Forintek Canada Corp in Vancouver. As shown in the figure, concentrically trussed braced timber frames were assumed at both ends of the multi-bay portal frame. To simplify the structure for computer analysis, only the concentrically trussed braced frame representing the main lateral load resisting system of the portal frame was analyzed (Figure 6.2).

![Figure 6.2. Concentrically trussed braced timber frame as the basic lateral load resistant system.](image)

The braced frame (Figure 6.2) was assumed to be 2.5 m wide and 6 m high. The columns were assumed to be manufactured out of one 6 m long glulam member with a cross section large enough to resist the vertical loads generated from gravity and seismic loads. There were three
diagonal braces and three cross beams placed between columns along the height of the frame. Dimensions of the portal frame shown in Figure 6.1 other than those of the braced frame alone, are given for orientation purposes only. For the analyses, two parameters were varied, namely the mass of the tributary roof area per braced frame, and the connection hysteresis curves. The gravity loads were assumed to be carried entirely by the columns. The lateral load was resisted by a discrete number of braced frames, the placement of which and number would determine the lateral load, generated by the acceleration of the tributary roof mass.

Figure 6.3. Elevation of a typical commercial type of braced frame building used in analysis.

The second structure analyzed was a typical three-storey commercial or residential type building with an elevation shown in Figure 6.3. Two of the five bays of the frame were braced, forming two concentrically trussed braced frames as main lateral load resisting system. Again, to simplify the structure for computer analysis, only one trussed bay, as the main lateral load resisting system of the portal frame, was analyzed. The braced frame modeled in this case was very similar to that presented in Figure 6.2, with a difference that the total height of the frame was $3 \times 2.5 = 7.5$ meters.
Both braced timber frames were modeled using the DRAIN-2DX (Powell, 1993) computer package for two-dimensional non-linear analysis of building structures. The analytical model representing the braced timber frame of the industrial type building is shown in Figure 6.4 a, while the model for the three storey commercial building is shown in Figure 6.4 b. In the first model, typical for warehouse structures (Figure 6.4a), the mass of the entire structure was concentrated at the roof level, equally distributed between the upper two nodes in the model. For that reason, this model will be referred to as "single-storey model" in the remainder of the chapter. The same principle of lumped mass was used in the second "multi storey model" (Figure 6.4b) where a mass was placed at every floor level, equally distributed between the nodes at that level.

Figure 6.4. Non-linear mathematical models in DRAIN-2DX used in analyses.
The frames in both models were assumed to be constructed of spruce-pine-fir (SPF) glued-laminated timber with material properties as determined from the material tests presented in previous chapters (Average MOE of 10,844 MPa and relative density of 0.405). The cross-sectional dimensions of the braces were the same (132 mm by 150 mm) as for all the other braces and connections throughout the study. All brace and crossbeam connections in the models were assumed to be pinned, as were the base support connections. Since the capacity design principle was used for the design of each frame, the nonlinearity of the model was concentrated in the braces, while the vertical columns and the horizontal beams were assumed to remain linear elastic.

Based on the results obtained from the cyclic connection tests and single brace shake table tests, non-linear mathematical models for all tested connections were defined. The “Florence” non-linear model (Ceccotti et al. 1989), was found to be very suitable for modeling the hysteretic behaviour of the brace connections. The subroutine for this model, as mentioned before, was incorporated in the DRAIN-2DX computer program. It is defined with a total of nine different parameters (Figure 5.2) and was described in detail in chapter five. It has produced satisfactory results when used for the prediction of the non-linear response of timber connections and elements in several research studies, including this one. Figure 6.5 presents the hysteresis loop of a riveted connection obtained during a cyclic test, compared with the analytical response of the same connection modeled with the Florence model and subjected to the same displacement history. It is evident that reasonably good representation of the connection load-displacement curve can be achieved.
Analytical models were developed for all seven tested connections. Safety factors throughout the analytical study were used as specified in the NBCC, a factor of 1.0 for earthquake loading on the load side and a $\phi$ factor of 0.6 on the resistance side of the code equation. To make a further adjustment for safety on the resistance side of the equation, the stabilized (third cycle) envelope curve from the cyclic tests was taken as the basis for defining the connection models. Previous research on cyclic behaviour of timber riveted connections on a large number of replicates (Karacabeyli et al., 1993), showed that the stabilized envelope curve can be taken as a representative for the 5th percentile connection behaviour, determined by assuming a 10% coefficient of variation and a normal distribution. The findings on the influence of dynamic rate of loading on the stiffness and ductility for different connections obtained from the shake table tests, were also taken into account when defining the model parameters for the final "characteristic" envelope for every connection type. Calibration of the connection model in each case was done not only in terms of strength and stiffness, but also in terms of energy dissipation. Model parameters were chosen so that the total hysteretic energy (area within the hysteresis loops) calculated from the analytical response of the stabilized loops matched the hysteretic energy dissipated during the stabilized loops from the cyclic tests at various deformation levels.

Figure 6.5. Tested and analytically obtained hysteresis behaviour of a riveted connection.
The analytical connection models were incorporated into non-linear analytical brace models using the serial spring procedure described in chapter five. The results from single brace shake table tests showed that when a braced frame is subjected to seismic loads, one connection of the brace will experience more deformation than the other. Because of this, the envelope of the final analytical brace model was defined as a combination of the characteristic envelopes of the weaker and stronger connection, and the linear elastic behaviour of the wood member (Figure 6.6). Based on observations from the shake table tests for four of the seven connections used in the analysis, the characteristic envelope of the stronger connection was assumed to reach the maximum load at half the deformation level of the weaker one. The ultimate deformation of the stronger connection was also assumed to be half of that of the weaker connection.

![Figure 6.6. Defining the analytical model for overall brace behaviour.](image)

Non-linear analytical brace models were defined for all seven connection types. The brace models were then used to simulate the brace elements in both single storey and multi-storey braced frame models (Figure 6.4). Using DRAIN 2DX, a series of non-linear time history dynamic analyses were performed for both frame models to determine their seismic performance and obtain an estimate of the force modification factors for these types of braced timber frames. The lateral load was related to the R-factor as described in section 6.4.1.
6.3 EARTHQUAKE GROUND MOTIONS

Dynamic time history analyses were conducted using five different acceleration records from previous earthquakes around the world. Records were chosen to satisfy the seismic zonal parameters of the National Building Code of Canada (NBCC) for a locality such as Vancouver, with peak horizontal ground velocity of 0.21 m/sec and peak horizontal ground acceleration of 0.23g, with probability of exceedance of 10% in 50 years. The records were denoted as VAN-29, VAN-48, VAN-68, VAN-72, and VAN-83 (Table 6.1), for use in the analyses.

Table 6.1. Main characteristics of the acceleration records used in analyses.

<table>
<thead>
<tr>
<th>Record Name</th>
<th>Peak Ground Velocity (m/s)</th>
<th>Peak Ground Acceleration (g)</th>
<th>PGA / PGV ratio</th>
<th>Seismic Event</th>
<th>Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>VAN-29</td>
<td>0.213</td>
<td>0.23</td>
<td>1.08</td>
<td>San Fernando 1971</td>
<td>6.4</td>
</tr>
<tr>
<td>VAN-48</td>
<td>0.235</td>
<td>0.23</td>
<td>0.98</td>
<td>San Fernando 1971</td>
<td>6.4</td>
</tr>
<tr>
<td>VAN-68</td>
<td>0.245</td>
<td>0.23</td>
<td>0.94</td>
<td>Montenegro 1979</td>
<td>6.4</td>
</tr>
<tr>
<td>VAN-72</td>
<td>0.193</td>
<td>0.23</td>
<td>1.19</td>
<td>Montenegro 1979</td>
<td>7.0</td>
</tr>
<tr>
<td>VAN-83</td>
<td>0.227</td>
<td>0.23</td>
<td>1.01</td>
<td>Loma Prieta 1989</td>
<td>7.1</td>
</tr>
</tbody>
</table>

The records in Table 6.1 are presented as they were used in the analyses, all of them scaled to have a peak ground acceleration of 0.23g and the corresponding velocity. The five earthquakes were chosen from a larger set of 22 different records that satisfy the zonal parameters from NBCC for Vancouver (Table 6.2), based on the analyses of their response spectra and the ductility demands exhibited during the preliminary dynamic analyses on a SDOF system. Some of the records from this set were recently used for the seismic upgrade analyses for the Oak Street Bridge in Vancouver. The records were from four real earthquakes around the world, namely the 1971 San Fernando (SF) California Earthquake (magnitude M=6.4), 1979
Montenegro (MN) Earthquake (magnitude M=7.0), 1986 Taiwan (TW) Earthquake (magnitude M=6.5) and 1989 Loma Prieta (LP) California Earthquake with magnitude M=7.1.

Table 6.2. Main characteristics of the larger set of records used for preliminary analyses.

<table>
<thead>
<tr>
<th>Record name</th>
<th>Event</th>
<th>Epicent. Distance (km)</th>
<th>Peak Horizontal Ground Motions</th>
<th>Station</th>
<th>Soil</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>a (g)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>V (m/s)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>VAN-04</td>
<td>SF</td>
<td>41</td>
<td>0.172</td>
<td>LAWP Office Building</td>
<td>Soft Rock</td>
</tr>
<tr>
<td>VAN-06</td>
<td>SF</td>
<td>41</td>
<td>0.152</td>
<td>222 Figueroa Street</td>
<td>Alluvium</td>
</tr>
<tr>
<td>VAN-08</td>
<td>SF</td>
<td>41</td>
<td>0.192</td>
<td>234 Figueroa Street</td>
<td>Alluvium</td>
</tr>
<tr>
<td>VAN-21</td>
<td>SF</td>
<td>70</td>
<td>0.044</td>
<td>6074 Park Drive, Wrightwood</td>
<td>Soil</td>
</tr>
<tr>
<td>VAN-26</td>
<td>SF</td>
<td>67</td>
<td>0.041</td>
<td>Palos Verdes Estates *</td>
<td>V=360 m/s</td>
</tr>
<tr>
<td>VAN-29</td>
<td>SF</td>
<td>42</td>
<td>0.114</td>
<td>900 S. Fremont Street</td>
<td>Soil</td>
</tr>
<tr>
<td>VAN-33</td>
<td>SF</td>
<td>33</td>
<td>0.180</td>
<td>Griffith Park Observatory</td>
<td>Hard Rock</td>
</tr>
<tr>
<td>VAN-35</td>
<td>SF</td>
<td>39</td>
<td>0.165</td>
<td>3407 West Sixth Street</td>
<td>Alluvium</td>
</tr>
<tr>
<td>VAN-42</td>
<td>SF</td>
<td>39</td>
<td>0.150</td>
<td>3710 Wilshire</td>
<td>Soft Rock</td>
</tr>
<tr>
<td>VAN-43</td>
<td>SF</td>
<td>30</td>
<td>0.150</td>
<td>3838 Lankevshim Blvd.</td>
<td>Soft Rock</td>
</tr>
<tr>
<td>VAN-48</td>
<td>SF</td>
<td>35</td>
<td>0.159</td>
<td>4867 Sunset Blvd.</td>
<td>Alluvium</td>
</tr>
<tr>
<td>VAN-50</td>
<td>SF</td>
<td>33'</td>
<td>0.113</td>
<td>3831 E. Ninth Street</td>
<td>Soil</td>
</tr>
<tr>
<td>VAN-51</td>
<td>SF</td>
<td>37</td>
<td>0.185</td>
<td>Millikan Library CIT</td>
<td>Soil</td>
</tr>
<tr>
<td>VAN-57</td>
<td>SF</td>
<td>37</td>
<td>0.188</td>
<td>450 N. Roxbury</td>
<td>Alluvium</td>
</tr>
<tr>
<td>VAN-63</td>
<td>MN</td>
<td>49</td>
<td>0.032</td>
<td>Podgorica, Seismic Station</td>
<td>No data</td>
</tr>
<tr>
<td>VAN-64</td>
<td>MN</td>
<td>49</td>
<td>0.031</td>
<td>Podgorica, Seismic Station</td>
<td>No data</td>
</tr>
<tr>
<td>VAN-65</td>
<td>MN</td>
<td>40'</td>
<td>0.040</td>
<td>Podgorica, Seismic Station</td>
<td>No data</td>
</tr>
<tr>
<td>VAN-68</td>
<td>MN</td>
<td>49</td>
<td>0.031</td>
<td>Ulcinj, Hotel Olimpik</td>
<td>Soil</td>
</tr>
<tr>
<td>VAN-72</td>
<td>MN</td>
<td>31</td>
<td>0.080</td>
<td>Herceg Novi</td>
<td>Rock</td>
</tr>
<tr>
<td>VAN-77</td>
<td>TW</td>
<td>67</td>
<td>0.189</td>
<td>Site E-02</td>
<td>Alluvium</td>
</tr>
<tr>
<td>VAN-82</td>
<td>LP</td>
<td>47</td>
<td>0.386</td>
<td>Palo Alto Hospital Basement</td>
<td>Soil</td>
</tr>
<tr>
<td>VAN-83</td>
<td>LP</td>
<td>51</td>
<td>0.288</td>
<td>Stanford, SLAC Lab *</td>
<td>V=360 m/s</td>
</tr>
</tbody>
</table>

* Soft soil conditions
Figure 6.7. Acceleration histories of the records used in the analyses.
The acceleration time histories of the five earthquake ground motions used in the analyses are presented in Figure 6.7. The pseudo acceleration response spectra of these records for five percent damping, compared to the Vancouver design spectrum are shown in Figure 6.8. As shown in the figures, besides satisfying the NBCC requirements for acceleration and velocity, the chosen records represent earthquakes with different frequency characteristics throughout the spectrum, recorded on different soil conditions.

Figure 6.8a. Pseudo acceleration response spectra for the records used in the analyses.
The main emphasis of the analyses presented here was to evaluate the force modification factors (R-factors) for braced timber frames in the National Building Code of Canada (NBCC). Force modification factors are assigned for different structural systems in NBCC and reflect the structure’s ability to undergo inelastic deformations and dissipate the seismic input energy. The higher the R-factor assigned for a certain structure, the higher the ability for energy absorption, resulting in a lower seismic design force given in the Code. According to NBCC, braced timber frames with “ductile connections” are assigned an R-factor of two, while all other braced timber

Figure 6.8b. Pseudo acceleration response spectra for the records used in the analyses.

6.4 NON-LINEAR TIME HISTORY DYNAMIC ANALYSES

The main emphasis of the analyses presented here was to evaluate the force modification factors (R-factors) for braced timber frames in the National Building Code of Canada (NBCC). Force modification factors are assigned for different structural systems in NBCC and reflect the structure’s ability to undergo inelastic deformations and dissipate the seismic input energy. The higher the R-factor assigned for a certain structure, the higher the ability for energy absorption, resulting in a lower seismic design force given in the Code. According to NBCC, braced timber frames with “ductile connections” are assigned an R-factor of two, while all other braced timber
frames are assigned an R-factor of 1.5. The criteria, however, for definition of a ductile versus non-ductile connection are not specified, neither in NBCC nor in CSA086.

6.4.1 Analysis Procedure

The non-linear analyses were carried out using both analytical DRAIN-2DX models described in section 6.2 (single-storey and multi-storey), each of them defined for all seven different brace connections. The industrial type “single storey” model will be used to explain the analysis procedure.

As mentioned in the second chapter, the NBCC expression for the minimum seismic base shear is given as:

\[ V = \frac{V_e}{R} \cdot U \] (6.1)

where \( V_e \) is the equivalent lateral seismic force representing elastic response, \( R \) is the force modification factor and \( U \) is a calibration factor representing a level of protection based on experience, with an assigned value of 0.6. To understand the implication of (6.1) it is useful to rearrange it in the following form:

\[ V \cdot \left( \frac{1}{U} \right) = \frac{V_e}{R} \] (6.2)

The factor \( 1/U \) can be considered as an over-strength factor. It has been observed that buildings designed using a base shear value of \( V \) have a lateral strength substantially higher than \( V \). The product of the base shear \( V \) and the over-strength factor \( 1/U \) gives an estimate of the actual lateral strength of the structure. The equation (6.2) simply states that the actual lateral strength of a structure is equal to the elastic strength demand \( V_e \), reduced by the \( R \)-factor. The elastic demand \( V_e \) can be expressed as:
\[ V_e = v \cdot S \cdot I \cdot F \cdot W \] (6.3)

where \( S \) is the seismic response factor, \( F \) is the foundation factor, \( I \) is the importance factor, \( W \) is the total seismic weight and \( v \) is a zonal velocity parameter. The seismic response factor \( S \) corresponds to a smoothed design acceleration response spectrum for 5% damped multiple degree of freedom system, subjected to an earthquake scaled to \( v = 1.0 \). The foundation factor \( F \) and the importance factor \( I \) in equation (6.3) account for the effects of local soil conditions and increase the safety level for post-disaster buildings, respectively.

The braced timber frame (model) analysed (Figure 6.9), was assumed to be located in Vancouver (\( v = 0.21 \)) with no post-disaster importance (\( I = 1.0 \)) and founded on a dense coarse-grained soil (\( F = 1.0 \)). The fundamental period of the frame \( T \) was determined to be 0.34 sec according to the NBCC recommendations using the formula:

\[ T = \frac{0.09 \cdot h}{\sqrt{D_s}} \] (6.4)
where \( h \) is the total height of the structure (\( h = 6.0 \) m) and \( D_s \) is the width of the braced frame (\( D_s = 2.5 \) m). It should be kept in mind that this expression is used for a large variety of structures and can at best be considered a crude estimate. More accurate values were obtained from Eigenvalue analyses described in section 6.4.2. For the specified period \( T \) and building location Vancouver (\( Za = Zv \)), the seismic response factor was calculated as \( S = 2.676 \). Having all the values needed, the design shear force \( V \) (Figure 6.9) can be calculated according to the formula (6.1) as:

\[
V = \frac{0.3372 \cdot W}{R} \tag{6.5}
\]

If we denote \( F_d \) as the design force in the brace which is at angle \( \alpha = 38.6^\circ \) (\( \cos \alpha = 0.78 \)) with the corresponding horizontal seismic design force \( V \), the equation (6.5) can be rearranged as:

\[
W = 2.31 \cdot R \cdot F_d \tag{6.6}
\]

The equation (6.6) expresses the value of the total seismic weight \( W \) as a function of the \( R \)-factor and the design force of the brace connections. Using equation (6.6), the mass \( M \) placed on the top of the analytical model (Figure 6.4a) can be calculated for a model with a certain connection type (known \( F_d \)) and an assigned \( R \)-factor.

A series of non-linear dynamic analyses was performed on models with seven different connections and with a range of \( R \)-factors between 1.25 to 3.5. The analyses were performed using a Newmark constant acceleration scheme in DRAIN-2DX, with a time step of 0.002 sec. Mass and stiffness proportional (Rayleigh) damping was included in all analyses with a value of 3% of critical damping and the P-delta effects were considered in all analyses. According to NBCC recommendations, the flexibility of the roof diaphragm in its own plane was not taken into account.
6.4.2 Fundamental Period

The response of the warehouse model closely resembled a single degree of freedom system, and the fundamental period would thus be a representative seismic design parameter for this type of braced frame building. The fundamental mode shape is presented in Figure 6.10. The fundamental period determined by the NBCC guidelines ($T = 0.34$ sec) would result in a conservative design, since the periods of the model obtained from Eigenvalue analyses for different R-factors were longer than the NBCC value for all cases. The fundamental periods of the single storey braced frame model calculated with the influence of the different connection stiffness and different R-factors are given in Table 6.3.

![Figure 6.10. Fundamental mode shape of the single storey analytical model.](image)

<table>
<thead>
<tr>
<th>Connection</th>
<th>Test Group</th>
<th>Fundamental Periods (sec) for Different R-Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>R = 1.0</td>
<td>R = 1.5</td>
</tr>
<tr>
<td>9.5 mm Bolts</td>
<td>I</td>
<td>0.41</td>
</tr>
<tr>
<td>12.7 mm Bolts</td>
<td>I</td>
<td>0.45</td>
</tr>
<tr>
<td>19 mm Bolts</td>
<td>I</td>
<td>0.44</td>
</tr>
<tr>
<td>Rivets</td>
<td>I</td>
<td>0.42</td>
</tr>
<tr>
<td>9.5 mm Bolts</td>
<td>II</td>
<td>0.36</td>
</tr>
<tr>
<td>12.7 mm Bolts</td>
<td>II</td>
<td>0.40</td>
</tr>
<tr>
<td>Rivets</td>
<td>II</td>
<td>0.34</td>
</tr>
</tbody>
</table>
6.4.3 Non-Linear Response Parameters

Typical examples of the seismic response parameters obtained from the analyses are shown in Figures 6.11 and 6.12. In Figure 6.11, the time histories of the acceleration and displacement at the top of the frame with riveted connections from the first test group, designed with an R-factor of 2.25, are given. The frame was subjected to the VAN-29 acceleration record with a maximum acceleration of 0.23 g. Figure 6.12 shows the load-deformation relationship of the bottom brace of the same frame with riveted connections designed with an \( R = 2.25 \) and subjected to the same earthquake record.

![Top Acceleration History - Riveted Connection I](image1)

![Top Displacement History - Riveted Connection I](image2)

Figure 6.11. Selected time histories of the response at the top of the braced frame model.

Because lateral forces are resisted by the axial forces in the braces, braced timber frames are regarded as relatively stiff structures. The fundamental periods shown in Table 6.3 and the
simple analysis of the typical response parameters such as those shown in Figure 6.11 suggested, however, a much softer nature of this structural system. By simple analysis of the displacement history at the top of the frame shown in Figure 6.11, it was observed that the vibration period of the frame shifted from approximately $T=0.6$ sec. initially to approximately $T=1$ sec. by the tenth second of the response. At this stage the frame has experienced its largest displacement and wood crushing and non-linear deformations had occurred in the connections.

Figure 6.12. Load-deformation response of the bottom brace for frame with riveted connections.

### 6.5 FORCE MODIFICATION FACTORS

Similar analyses were done for all the frame models, each subjected to the five different earthquake records. As a basis for evaluation, the maximum displacements in the brace connections were extracted. Results from the non-linear dynamic analyses (regarding the estimate on the force modification factors) are summarised in graphs as shown in Figures 6.13 to 6.16. Each graph presents results for a braced frame with a different connection detail subjected to a total of five different earthquake records. Variation of the R-factor, as explained
earlier in the analytical procedure, was achieved by changing the roof tributary area (and thus the mass), while keeping the braced frame members and brace connections the same.

![Deformation Demand for Frames with 12.7 mm Bolted Connection I](image1)

**Figure 6.13.** Deformation demands of braced timber frames with 12.7 mm bolted connections.

Each graph represents the deformation demand (Y-axis) of the bottom brace of a braced frame model designed with a certain R-factor according to NBCC (X-axis), for each of the five different earthquakes. The two horizontal lines represent the yield and the ultimate deformation capacity of the brace, respectively. Because braced timber frames as a system typically have little redundancy, it was assumed that the structure has failed when the deformation demand of
at least one given earthquake exceeded the capacity of the brace connections. As shown in Figure 6.13, the braced frame model with 12.7 mm (1/2 in) bolted connections from the first test group would survive all five earthquakes if designed with an R-factor lower than 2.0, using the CEN definition of failure (at 0.8 $P_{\text{max}}$ after ultimate). On the other hand, the same frame, when built with the same diameter bolts but in arrangement as given for the second test group could survive all the earthquakes if designed with an R-factor of up to 1.75. Similar observations can be made for frames with other connections from both test groups shown in Figures 6.13 to 6.16.

![Deformation Demand for Frames with 9.5 mm Bolted Connection I](image1)

![Deformation Demand for Frames with 9.5 mm Bolted Connection II](image2)

Figure 6.14. Deformation demands of braced timber frames with 9.5 mm bolted connections.
Figure 6.15. Deformation demands of braced timber frames with riveted connections.

R-factor values corresponding to the worst case, out of the five selected earthquakes, for each connection type are summarised in Table 6.4. Two sets of R-factors were calculated, depending on the definition of ultimate displacement in a connection. According to the CEN Standard, the ultimate displacement capacity of all connections can be determined at a point where the load has dropped to 80% of the maximum load. This definition for ultimate deformation was used in Figures 6.13 to 6.16. Test results showed, however, that in bolted connections splitting always occurred at the maximum load, so it could be argued that the displacement at this point should
be defined as the ultimate displacement. If this failure criterion is used, then the maximum R-factors for which the structure will survive all earthquakes are given in the last row of Table 6.4. This criterion was not applied to the riveted connections because they do not experience any splitting effects. It should be noted, however, that in both cases the assessment of a structure not failing does not include any safety evaluation in terms of probability.

Figure 6.16. Deformation demands of a braced timber frame with 19 mm bolted connections.

Table 6.4. Maximum R-factors for braced timber frames with different connections.

<table>
<thead>
<tr>
<th>Fastener</th>
<th>9.5 mm Bolts</th>
<th>12.7 mm Bolts</th>
<th>19 mm Bolts</th>
<th>Rivets</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test group</td>
<td>I</td>
<td>I</td>
<td>I</td>
<td>I</td>
</tr>
<tr>
<td>R-factor (at 80 %)</td>
<td>1.75</td>
<td>1.5</td>
<td>2.0</td>
<td>1.75</td>
</tr>
<tr>
<td>R-factor (max load)</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
</tr>
</tbody>
</table>

It is evident that the earthquake record VAN-83 was critical for all seven cases. The influence of this record on the response of the frames was significant because of the high spectral amplitudes for longer periods (Figure 6.8), which are much higher than the amplitudes given in the NBCC design spectrum for a site such as Vancouver. This long period range (from 0.5 to 1.2 sec) corresponds to the range of natural periods typical for braced timber frames during shaking. As
mentioned before, contrary to common belief that braced timber frames are quite stiff structures, their initial natural periods were found to be longer than expected (0.4 to 0.85 sec), depending on the connection detail and the R-factor used for design. The initial periods are being significantly prolonged during the actual seismic event, because of the loosening of the connections after the first few larger cycles of motion. This suggests that braced timber frames can be expected to behave relatively poorly if built on softer soils and this should be taken into account during the design, along with the higher value for the soil factor F.

The results also suggest that braced frames with different connections should be assigned different R-factors. For connections with mild steel bolts with slenderness ratio \( l/d \) (where \( d \) is the bolt diameter and \( l \) is the width of the wood member) greater than ten, an R-factor of 1.5 to 2.0 appears to be reasonable. Frames with mild steel bolted connections with \( l/d \) ratio lower than ten should be treated as non-ductile and the currently prescribed R-factor of 1.5 might even be unconservative. On the other hand, the current R-factor of 2.0 appears to be appropriate for frames with riveted connections, although a relatively low margin of safety was noticed for that particular R-factor in frames with riveted connections from the first test group.

The results from the analyses can be interpreted also in terms of the maximum seismic weight \( W \) that is allowed for a braced frame with certain connections. Graphs similar to those presented in Figures 6.13 to 6.16 can be developed, which show the relationship of deformation demand on the brace for frames with different connections and the \( W/F_d \) ratio for each of the five earthquakes. Figure 6.17, for instance, shows such graphs for braced frames with 9.5 mm bolted connections and riveted connections from the first test group. The results in the figure show that a braced timber frame with 9.5 mm bolted connections should be allowed maximum seismic
weight of 4 to 4.25 times the design force in the brace, while the frame with riveted connections can be allowed up to 4.75 times the design force. For ductile bolted connections this value should not exceed 4.0, while for riveted connections a value of 4.5 is considered appropriate.

Figure 6.17. Brace demand as function of $W / F_{d}$ ratio for frames with different connections.

### 6.5.1 Influence of Frame Geometry on R-Factors

To assess the influence of different braced frame geometry on the R-factor values, another series of dynamic time history analyses was performed. A total of five braced frames with different
aspect ratios were analysed, referred to as case I to case V in the remainder of this section. The frame models analysed were again six meters high and had three braces placed along the height, identically as the model used so far. The width of the frame \( D_s \) (Figure 6.9) was assumed different for each case, starting from 1.5 m for case I to 3.5 m for case V, with the aspect ratios (height to width) from 1.33 to 0.57 respectively (Table 6.5). The frame model which was used in the analyses so far (\( D_s = 2.5 \) m) is thus referred to as case III in this set of analyses.

Table 6.5. Basic geometry information for the frame models used in analyses.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Case I</th>
<th>Case II</th>
<th>Case III</th>
<th>Case IV</th>
<th>Case V</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame Height ( H ) (m)</td>
<td>6.0</td>
<td>6.0</td>
<td>6.0</td>
<td>6.0</td>
<td>6.0</td>
</tr>
<tr>
<td>Frame Width ( D_s ) (m)</td>
<td>1.5</td>
<td>2.0</td>
<td>2.5</td>
<td>3.0</td>
<td>3.5</td>
</tr>
<tr>
<td>Aspect Ratio ( h/D_s )*</td>
<td>1.33</td>
<td>1.0</td>
<td>0.8</td>
<td>0.67</td>
<td>0.57</td>
</tr>
<tr>
<td>Brace Angle ( \alpha ) (deg)</td>
<td>53.1</td>
<td>45.0</td>
<td>38.6</td>
<td>33.7</td>
<td>29.7</td>
</tr>
<tr>
<td>Brace Length ( L ) (m)</td>
<td>2.50</td>
<td>2.82</td>
<td>3.20</td>
<td>3.60</td>
<td>4.03</td>
</tr>
</tbody>
</table>

* \( h = H/3 \)

Non-linear analytical models defining the overall brace behaviour for all seven different connections (Figure 6.6) were defined again for each different frame (different brace length) representing a different case. Then the analysis procedure described in section 6.4.1 was performed for all different cases of frames, defining the relationship between the seismic weight and the design R-factor. Fundamental periods for all frames were determined and based on the mass and stiffness damping coefficients calculated to represent a damping value of 3% of critical. Fundamental periods for frames designed with 12.7 mm bolted and riveted connections of the second tests group, and different R-factors and aspect ratios are shown in Table 6.6.

To reduce the number of dynamic analyses, only three earthquake records were used this time namely VAN-83, VAN-68, and VAN-48. Results from the analyses can be summarised in similar graphs as those shown so far. For example, Figure 6.18 shows the deformation demand
for five cases of frames with 12.7 mm bolted and riveted connections from the second test group, subjected to VAN-68 earthquake record.

Table 6.6. Fundamental periods for frame models with different aspect ratios.

<table>
<thead>
<tr>
<th>Connection</th>
<th>Frame Case</th>
<th>R = 1.0</th>
<th>R = 1.5</th>
<th>R = 2.0</th>
<th>R = 2.5</th>
<th>R = 3.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.7 mm Bolts</td>
<td>I</td>
<td>0.62</td>
<td>0.72</td>
<td>0.83</td>
<td>0.93</td>
<td>1.02</td>
</tr>
<tr>
<td>Test Group II</td>
<td>II</td>
<td>0.47</td>
<td>0.55</td>
<td>0.63</td>
<td>0.70</td>
<td>0.77</td>
</tr>
<tr>
<td></td>
<td>III</td>
<td>0.40</td>
<td>0.49</td>
<td>0.57</td>
<td>0.64</td>
<td>0.70</td>
</tr>
<tr>
<td></td>
<td>IV</td>
<td>0.38</td>
<td>0.47</td>
<td>0.54</td>
<td>0.61</td>
<td>0.67</td>
</tr>
<tr>
<td></td>
<td>V</td>
<td>0.37</td>
<td>0.46</td>
<td>0.53</td>
<td>0.59</td>
<td>0.65</td>
</tr>
<tr>
<td>Rivets</td>
<td>I</td>
<td>0.53</td>
<td>0.62</td>
<td>0.71</td>
<td>0.79</td>
<td>0.87</td>
</tr>
<tr>
<td>Test Group II</td>
<td>II</td>
<td>0.39</td>
<td>0.46</td>
<td>0.54</td>
<td>0.60</td>
<td>0.66</td>
</tr>
<tr>
<td></td>
<td>III</td>
<td>0.34</td>
<td>0.43</td>
<td>0.49</td>
<td>0.55</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td>IV</td>
<td>0.33</td>
<td>0.42</td>
<td>0.48</td>
<td>0.54</td>
<td>0.59</td>
</tr>
<tr>
<td></td>
<td>V</td>
<td>0.30</td>
<td>0.39</td>
<td>0.45</td>
<td>0.51</td>
<td>0.56</td>
</tr>
</tbody>
</table>

As shown in the figure, the aspect ratio had an influence on the deformation demand experienced during the earthquake. The differences, however, were much smaller than was previously anticipated. For bolted connections, for instance, the differences in demand were minimal for frame cases II to V in the design range of R-factor up to 2.0. The differences were greater for R-factors higher than two but this has no significance because it is outside the maximum recommended design value for R. Case I frames, however, due to their higher period of vibration showed a significant increase in demand compared to the other four cases. Frames with riveted connections showed similar behaviour as well. The overall result patterns were also found similar for frames with other connections for that particular earthquake.

Maximum values for R-factors for which frames with 12.7 mm bolted and riveted connections (second test group) with different aspect ratios will survive all the earthquakes, are given in Table 6.7. The results show that narrow braced frames (with aspect ratios higher than one)
should be avoided, because of their cantilever bending type response. Wider frames exhibit more of a shear type response and make better use of the braces and their connections. Very wide frames with aspect ratios lower than 0.67, however, should also be avoided because the benefits of having a wider frame are usually outweighed by the drawbacks of having a long brace, susceptible to buckling at lower force levels. Finally, the maximum R-factors for frames with different connections were found not to be significantly affected by the frame aspect ratio within the recommended aspect range of 0.67 to 0.9.

Frames with 12.7 mm Bolted Connection II - VAN-68 Record

Frames with Riveted Connection II - VAN-68 Record

Figure 6.18. Deformation demand for frames with different aspect ratios for VAN-68 record.
Table 6.7. Maximum R-factors for frames with different aspect ratios.

<table>
<thead>
<tr>
<th>Connection</th>
<th>Frame Case</th>
<th>Maximum R-Factors at 80 % $F_{\text{max}}$</th>
<th>Maximum R-Factors at $F_{\text{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.7 mm Bolts</td>
<td>I</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Test Group II</td>
<td>II</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>III</td>
<td>1.75</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>IV</td>
<td>1.75</td>
<td>1.75</td>
</tr>
<tr>
<td></td>
<td>V</td>
<td>1.75</td>
<td>1.75</td>
</tr>
<tr>
<td>Rivets</td>
<td>I</td>
<td>1.25</td>
<td>-</td>
</tr>
<tr>
<td>Test Group II</td>
<td>II</td>
<td>1.75</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>III</td>
<td>2.0</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>IV</td>
<td>2.0</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>V</td>
<td>2.0</td>
<td>-</td>
</tr>
</tbody>
</table>

6.5.2 Period Dependency of R-factors

The period dependency of force modification factors has been investigated by many researchers. The trend recognised from this investigations is that for structures with moderate and long structural periods, the reduction factor is related to the equal displacement principle, while for structures in the short period range it is related to the equal energy principle. In this section of the thesis, an effort was made to determine the period dependency of the ductility demands for braced timber frames with different connections and designed with different R-factors.

The procedure for determining the period dependency of ductility demands included the establishment of single degree of freedom models that represent the overall frame behaviour. For that purpose, non-linear static (cyclic pushover) analyses were performed using the basic braced frame model (Case III) as presented in Figure 6.19. Analytical models for braced frames with all seven connection types were developed. To reduce the number of analyses, however,
only two models were used for dynamic analyses this time. A 12.7 mm bolted and riveted connection from the second test group were chosen as representative for typical bolted and riveted connections. During the non-linear static analyses, both frame models were subjected to a displacement input at the top, defined similarly as the testing protocol used earlier for cyclic tests of the corresponding connection (Figure 6.19).

Figure 6.19. Cyclic pushover analysis using the basic braced frame model.

The load-displacement hysteresis loops for the entire frames obtained from the non-linear static analyses, were then used to define the hysteretic characteristics of a single degree of freedom model for both bolted and riveted frames. The analytical procedure used further in the analyses was based on the procedure explained in section 6.4.1. Equation (6.6) expresses the value of total seismic weight \( W \) as a function of the \( R \)-factor and the connection design force \( F_d \) for a case III braced frame. If we denote \( f \) as the ratio between the connection design force \( F_d \) and the yield force (force at yield deformation) for the brace \( F_y \) as shown in (6.7),
and express the brace yield force $F_y$ in terms of the frame yield force $F_{yf}$, then the seismic weight can be expressed as:

$$W = 2.31 \cdot R \cdot f \cdot F_{yf} \cdot \frac{1}{\cos \alpha}$$  \hspace{1cm} (6.8)

If we substitute the value of the cosine of the angle for frame case III (0.78) and express the frame yield force in terms of initial stiffness $K_l$ and yield deformation $\Delta_y$, we get:

$$W = 2.96 \cdot R \cdot f \cdot K_l \cdot \Delta_y$$  \hspace{1cm} (6.9)

Equation (6.9) can be further developed for each frame with a particular connection. For instance, for a braced frame with riveted connections from the second test group, with the value for $f$ and $K_l$ implemented, the equation takes the form:

$$\Delta_y = \frac{0.185 \cdot W}{R}$$  \hspace{1cm} (6.10)

Equation (6.10) can now be used to determine the yield deformation as a function of the seismic weight $W$ and the $R$-factor used in the design. By choosing a value for the mass $W$ we also change the period of the structure (which can be calculated), so the results of the analyses can be expressed for each particular period of the structure. According to the procedure, the yield displacement of the SDOF model was changed in each analysis, according to the seismic weight $W$ and the $R$-factor, while the displacement at maximum load $\Delta_2$ was defined in terms of $\Delta_y$ based on the observations from experiments. Similarly, based on the research data, the force associated with zero deformation $P_0$ was assumed to be a constant percentage of the maximum force for each different connection.
These models were used in a number of non-linear dynamic time history analyses, to determine the period dependency of ductility demands for frames with different connections and designed with different R-factors. Figure 6.20 shows some of the results from the analyses, namely the period dependency of ductility demands for braced frames with 12.7 mm bolted connections (R=1.5) and riveted connections (R=2.0) for the three different earthquakes. The graphs presented were chosen for the maximum allowable R-factors for each connection according to the results shown in section 6.5.

Figure 6.20. Period dependency of ductility demand for selected braced timber frames.
Analyses showed that the ductility demand decreased as the period of the structures increased. Figure 6.20 shows that braced frames with bolted connections designed with $R=1.5$ should have a fundamental period larger than 0.25 sec in order to keep the demand lower than the capacity. In the period range between 0.4 and 0.8 sec it appears that the R-factor closely resembled the average demand, while for longer periods a higher R-factor could be used to account for the lower ductility demand. For riveted connections, the demand was found to be higher than the used R-factor of 2.0 for the expected period range for the frame of up to 0.8 sec.

Figure 6.21. Period dependency of average demand for frames with different connections.
Results from the analyses can also be presented in graphs such as those shown in Figure 6.21. The figure shows the period dependency of the average ductility demands (for all 3 earthquakes used) for the frames with bolted and riveted connections analysed. Figure 6.21 again shows that frames with bolted connections can be designed with a maximum R-factor of 1.5, while the frames with rivets can be designed with an R-factors of 2.0. Designing frames with higher R-factors would require the structure to have unrealistic fundamental period, to have the ductility demand lower than the capacity.

6.5.3 R-factors for Multi-Storey Braced Frames

Dynamic analyses were also performed on the three storey braced frame model of a commercial type building, previously shown in Figure 6.4b. The objective of the analyses was to determine the difference in maximum allowable R-factors for these types of braced frames and compare the results to the industrial type model. In order to allow comparison with the previous model, the same five acceleration records were used. As before, damping was also assumed as 3% of critical for all three modes contributing to the overall response. The total seismic weight W used in the model was also the same for each R-factor as for the industrial type model. However, three different cases of distribution of that mass along the height of the structure were considered. The mass distribution that determines the three cases is given in Table 6.8, while the graphic presentation is given in Figure 6.22. The three cases were chosen to represent the most common cases used in practice. In other words, the response of most cases was expected to fall within the limits of the mass distributions analysed here. The typical mode shapes for the first three modes for case I are shown in Figure 6.23. To reduce the number of analyses, brace models with R-factors from 1.5 to 2.25 were analysed for all three cases. For models where
lower R-factors were expected, such as the frame with 19 mm bolted connection, only analyses for R-factors lower than R=1.5 were performed.

Table 6.8. Storey mass distribution for all three model cases used.

<table>
<thead>
<tr>
<th>Storey (mass)</th>
<th>Storey mass in terms of seismic weight W</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Case I</td>
</tr>
<tr>
<td>I (M1)</td>
<td>0.333</td>
</tr>
<tr>
<td>II (M2)</td>
<td>0.333</td>
</tr>
<tr>
<td>III (M3)</td>
<td>0.333</td>
</tr>
</tbody>
</table>

As expected, the seismic response of the multi-storey models was different than of that of the industrial type model. The influence of the second mode of vibration, although contributing only about 10% of the total response, was noticeable in analyses for certain acceleration records (VAN-48 and VAN-72). This caused the maximum deformation demands to be experienced not only at the lowest brace but also at the upper ones. In addition, the VAN-83 earthquake was not the most critical record for all cases anymore, as for the single storey model. The influence of the third mode was found to be negligible. The maximum deformation demand at the brace
level, when compared to the single storey model, was found to vary based on the connection type and on the case of the frame. Figure 6.24 shows the maximum brace demands for the three cases of multi-storey frames with 12.7 mm bolted and riveted connections from the first test group, designed with an R-factor of 2.0 and compared to the corresponding single storey model.

![Mode Shapes](image)

Figure 6.23. First, second and a third mode shape of typical three-storey model - Case I.

As shown in the figure, the maximum demands for some of the frames were significantly different when compared to the demand for the single storey model (Figure 6.24a). For others (Figure 6.24b and Figure 6.25), the demands of the multi-storey model were found to be higher for all earthquakes and all frame cases. Generally, the demands for multi-storey models were either at the same level or higher than the corresponding single storey ones. In some cases the higher demands resulted in lower maximum allowed R-factors while in other, the differences were negligible. The maximum suggested R-factors for multi-storey braced frames with different connections are given in Table 6.9.
Figure 6.24. Deformation demands of multi-storey frames with bolted and riveted connections.

Table 6.9. Maximum R-factors for multi-storey braced frames with different connections.

<table>
<thead>
<tr>
<th>Fastener</th>
<th>9.5 mm Bolts</th>
<th>12.7 mm Bolts</th>
<th>19 mm Bolts</th>
<th>Rivets</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test group</td>
<td>I</td>
<td>II</td>
<td>I</td>
<td>II</td>
</tr>
<tr>
<td>R-factor (at 80 %)</td>
<td>1.75</td>
<td>1.5</td>
<td>2.0</td>
<td>1.75</td>
</tr>
<tr>
<td>R-factor (max load)</td>
<td>1.5</td>
<td>1.25</td>
<td>1.5</td>
<td>1.5</td>
</tr>
</tbody>
</table>

As shown in the table, the maximum R-values either were the same or lower than those obtained from the single storey model. They were still, however, within the suggested R-values of 1.5 to
2.0 for frames with slender bolts and 2.0 for frames with rivets. Case II was most often found to be the most critical one of the three different cases of frames. To account for the lower R-factors obtained from these analyses, a maximum R-factor of 1.5 may be suggested for braced frames with slender bolts and a factor of 2.0 for rivets. The results also showed that stocky bolts should be avoided in braced timber frames.

![Deformation demands for single versus multi-storey frames](image)

Figure 6.25. Deformation demands of single and multi-storey frames with riveted connections.

### 6.6 SUMMARY

Results from the non-linear analyses on seismic performance and force modification factors for braced timber frames are presented in this chapter. Braced frame models of a single storey industrial type building and multi-storey commercial type building were analysed. For each model, load deformation characteristics of seven different connection details were used, based on the results of quasi-static and shake table tests.
It is evident from the presented results that the seismic response of braced timber frames is heavily influenced by the behaviour of the connections. The results suggest that braced frames should be assigned different R-factor depending on the connection type. Braced timber frames with mild steel (ASTM A-307) bolted connections with slenderness ratios \((l/d)\) of 10 or higher exhibited far more adequate seismic performance than frames with lower bolt slenderness ratios. Further research is needed, however, to study the effect of other parameters such as end distance, spacing, number of rows, number of bolts in a row etc., on the seismic performance of the connections and frames in general. Until such research is undertaken, an R-factor of 1.5 appears to be reasonable for braced timber frames with slender bolts. On the other hand, glulam riveted connections showed promising results when used for braced timber frames. Braced frames with glulam riveted connections designed in rivet yielding mode may be assigned an R factor of 2.0, in recognition of their higher and more consistent ductility capacity. It should be noted, however, that the assessment of a structure not failing does not include any safety evaluation in terms of probability.
7. SHAKE TABLE TESTS ON A TWO-STOREY BRACED FRAME MODEL

Results from different tests and dynamic analyses aimed at determining the seismic performance of braced timber frames were presented in previous chapters of this thesis. Based on these results, many parameters influencing the seismic behaviour of braced timber frames were presented, and recommendations on force modification factors were made. Although this information may be assumed sufficient to characterize the seismic response of braced timber frames, no comprehensive research project is complete without one of the best ways to investigate a seismic performance of a structure - to conduct a shaking table test on an entire structural model. The lack of published results on performed shake table tests on braced timber frames was an additional challenge in pursuing the final goal.

For these reasons, a series of shake table tests on a two-storey braced timber frame model with riveted connections was conducted as at the next phase of the research program. Shake table tests were conducted in the Earthquake Engineering and Structural Dynamics Research Laboratory (Earthquake Laboratory) at the Department of Civil Engineering, University of British Columbia. The description of the dynamic testing facility at UBC was given in section 4.2, so it will not be repeated here. As mentioned before, the objectives of the shake table tests were to observe and assess the response of this particular braced frame to earthquake excitation and use the results later for verification of analytical simulations of the response of the frame.
7.1 DESCRIPTION OF THE MODEL

The two-storey braced frame model consisted of two planar concentrically trussed braced timber frames placed 120 cm apart. Each braced frame was 161 cm wide (E-W) and $2 \times 125 = 250$ cm high with a brace aspect ratio of 0.78, which was close to the case III frames analysed in the chapter six. When determining the dimensions of the model, various limitations such as shake table size, shake table capacity, laboratory height and availability of input records were taken into account. The frames were made of SPF glulam members 130x152 mm in cross-section, the same as for all other tests throughout this study. The two planar frames were braced together in the out-of-plane direction (N-S) with crossed steel angles L102x102x6.4. A picture of the braced frame model placed on the shake table and ready for testing is shown in Figure 7.1.

Figure 7.1. Braced frame model ready for the shake table tests (NE view).
The columns of the frame were continuous members along the height of the frame. All other elements (horizontal cross-beams and braces) were attached to the columns using 65 mm long glulam rivets and 6.4 mm thick steel side plates. Glulam riveted connections were used in construction of the frame because of their superior performance in the previous tests. The bottoms of the columns were attached to the shake table with special steel footings with hinge connections, allowing free rotation of each column in the plane of the motion. A 3-D drawing of the braced frame model is shown in Figure 7.2.

Figure 7.2. 3-D drawing of the braced frame model.

A 16.6 kN concrete block was attached to the top of the model. The block was bolted to the braced frame using lag screws and special cast-in steel brackets, placed on each side of the
Three additional steel blocks with a total weight of 13.4 kN were placed and fixed on top of the concrete block, making the total weight on the top to be 30 kN. Another three steel blocks with a total weight of 13.4 kN were placed on the first storey crossbeam. They were kept in place by steel angles placed on both sides and a special hold down connection. The weight of the blocks on both levels was partially supported by the glulam rivets in the connections and partially by steel seat angles below the cross beams, fixed to the column using six 90 mm long rivets.

A conservative approach was used for the design of the model, since braced frames do not have much redundancy and failure in the braces would lead to total collapse. A linear dynamic analysis of a model developed in SAP 2000 was used to check the peak force values in the members. Although the analysis was limited to the linear elastic response of the frame, this information was very useful to estimate the behaviour of the frame and ensure that the level of excitation provided by the shake table was sufficient to provide the desired response.

A capacity design procedure was used for the design of the braced frame model. The braces were connected to the rest of the frame by twelve rivets on each end, six on each face of the element. The other side of the connection was then designed to be able to sustain in the linear range the maximum force expected to be developed in the brace. In other words, the design force of the rest of the connection was made higher than the maximum force able to develop in the brace. Although the connection with twelve rivets was not tested in the quasi-static or shake table program, based on the experience from these tests the maximum force in the brace was estimated to be 60 % higher than the design force. Each connection was installed so that the minimum end and edge distances were satisfied according to CSA-O86. The rivets in the
connections of the cross beams to the columns were placed closely together near the center of the member to allow for development of rotations. In addition, sufficient space was left for brace connections to experience the expected deformations without the brace touching the rest of the frame. All rivets were driven with a hammer by hand in a pre-drilled holes in the steel plates.

The model members were cut from 3.6m (12 feet) SPF glulam beams. The properties of the beams were determined prior to the building of the model and are presented in section 4.7 of the thesis. The average properties found were 10,844 MPa for the MOE, a relative density of 0.405 and a moisture content of 15.2 %. The glued laminated members were conditioned prior to construction of the model in a laboratory environment at an average temperature of 20 ± 3° C and relative humidity of 50 % ± 10% for more than 12 months. This was deemed to be sufficient time for members to reach the equilibrium moisture content in dry service conditions. The riveted connections were conditioned for three weeks to allow for relaxation of the wood fibres around the rivets.

7.2 INSTRUMENTATION AND DATA ACQUISITION

A number of sensors were installed to monitor the dynamic behaviour of the braced frame model during the shake table tests. A total of 16 different time history signals (channels) of digital information were recorded during each test. The locations of the instruments used to record the signals on the south frame and north frame are shown in Figures 7.3 and 7.4 respectively. A list of the sensors and the measured parameters are given in Table 7.1.
Strong motion accelerometers were placed at two levels, one at the top of the model (Ch. 4) and one at the first storey (Ch. 3). Both accelerometers recorded accelerations in the E-W direction and had a range of measurement of ± 5g with a precision of 0.1%. In addition, a reading from the acceleration at the shake table level (Ch. 2) was taken using the built-in Kistler K-Beam accelerometer. For impact test purposes only, one additional accelerometer was placed at each storey (not shown) to record the out of plane acceleration (N-S). To characterise the model response on a structural level, four displacement position transducers were placed at the east side of the model. One sensor was placed at each storey at the east corner of both the north frame (Ch. 14 and Ch. 15) and the south braced frame of the model (Ch. 16 and Ch. 1). A record was also taken of the built in horizontal E-W displacement sensor (LVDT-Ch. 13) of the shake table.
The response of the model on a brace level was monitored by four displacement transducers, one for each of the four glulam braces (Ch. 5, Ch. 6, Ch. 7 and Ch. 8). The displacements were measured between the two steel plates at each end of the brace member. The connection slip, as a difference in displacement of the steel plate and the wood brace, was also measured at all four brace connections on the south frame of the model. Four displacement transducers (DCDTs) were mounted on the north side of the south frame only (Ch. 9, Ch. 10, Ch. 11 and Ch. 12). The displacement transducers were calibrated for a displacement range of ± 25.4 mm (1 inch) with an accuracy of 0.1 %.

The information was recorded using one of the data acquisition systems in the laboratory (Labview), with a sampling rate of 200 data points per second. All recorded signals were filtered with a 30 Hz low-pass filter using a 3 pole Bessel type filter with a 60 dB/decade roll off.
Following each test, some of the recorded channel outputs were viewed on a monitor to verify the data. This quick verification was essential in detection of some recording problems during the tests. Finally, a video camera was used to capture the response of the entire frame under the earthquake loading for each test.

Table 7.1. List of the sensors used during the shake table tests and their location.

<table>
<thead>
<tr>
<th>Location</th>
<th>Instrument</th>
<th>Measurement</th>
<th>Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Displacement Sensor</td>
<td>First Storey - SE Corner</td>
<td>Horizontal</td>
</tr>
<tr>
<td>2</td>
<td>Accelerometer</td>
<td>Shake Table Acceleration E-W</td>
<td>Horizontal</td>
</tr>
<tr>
<td>3</td>
<td>Accelerometer</td>
<td>First Storey Acceleration E-W</td>
<td>Horizontal</td>
</tr>
<tr>
<td>4</td>
<td>Accelerometer</td>
<td>Top Mass Acceleration E-W</td>
<td>Horizontal</td>
</tr>
<tr>
<td>5</td>
<td>Displacement Sensor</td>
<td>Second Storey, Entire North Brace</td>
<td>Diagonal</td>
</tr>
<tr>
<td>6</td>
<td>Displacement Sensor</td>
<td>First Storey, Entire North Brace</td>
<td>Diagonal</td>
</tr>
<tr>
<td>7</td>
<td>Displacement Sensor</td>
<td>Second Storey, Entire South Brace</td>
<td>Diagonal</td>
</tr>
<tr>
<td>8</td>
<td>Displacement Sensor</td>
<td>First Storey, Entire South Brace</td>
<td>Diagonal</td>
</tr>
<tr>
<td>9</td>
<td>DCDT</td>
<td>Top Connection II Storey South</td>
<td>Diagonal</td>
</tr>
<tr>
<td>10</td>
<td>DCDT</td>
<td>Bottom Connection II Storey South</td>
<td>Diagonal</td>
</tr>
<tr>
<td>11</td>
<td>DCDT</td>
<td>Top Connection I Storey South</td>
<td>Diagonal</td>
</tr>
<tr>
<td>12</td>
<td>DCDT</td>
<td>Bottom Connection I Storey South</td>
<td>Diagonal</td>
</tr>
<tr>
<td>13</td>
<td>Displacement Sensor</td>
<td>Shake Table Displacement E-W</td>
<td>Horizontal</td>
</tr>
<tr>
<td>14</td>
<td>Displacement Sensor</td>
<td>Top Frame NE Corner</td>
<td>Horizontal</td>
</tr>
<tr>
<td>15</td>
<td>Displacement Sensor</td>
<td>First Storey NE Corner</td>
<td>Horizontal</td>
</tr>
<tr>
<td>16</td>
<td>Displacement Sensor</td>
<td>Top Frame SE Corner</td>
<td>Horizontal</td>
</tr>
</tbody>
</table>
7.3 TESTING PROCEDURES

A total of 24 different tests were performed on the braced frame specimen. This included impact tests to determine the fundamental frequencies of the frame, low level shaking with different earthquake records and high level shaking to impose damage. A summary of all the tests is given in Table 7.3 later in the chapter.

7.3.1 Impact Hammer Tests

Four series of impact hammer tests were performed on the braced frame model. The first one was conducted at the beginning before the shake table tests, the second after the tests with 50% span, the third after the tests with 100% span during which significant non-linear deformations were recorded and the fourth at the end of the testing program. In each series, impact tests were conducted in both the in-plane E-W direction and the out-of-plane N-S direction.

There were several objectives for performing the impact hammer tests on the braced frame model. The primary objective of the initial tests was to obtain the frequency of the fundamental mode of vibration. It was desirable to know this frequency in order to choose appropriate acceleration records for the shake table tests. Also, by obtaining the in-plane and out-of-plane stiffness characteristics, an estimate could be made on the likelihood of expecting some torsional vibration of the model during the tests, so that steps could be taken to avoid the cross-coupling of modes. The objective of the other series of impact tests was to define the stiffness changes on the model due to non-linear deformations in the brace connections. In addition, all tests were very useful for calibration of the analytical model developed at the end of the study, and in determining the damping properties of the model.
The hammer tests were performed at the Earthquake Laboratory with the model already mounted on the shake table and ready for testing. An instrumented sledgehammer (Dytran model 5803A) was used for the impact testing. The hammer impacts were applied horizontally at the middle of the mass mounted at the top of the frame in both directions. The measured data during the impact tests included the force signal from the hammer and the three horizontal in-plane accelerations, namely at the top, at the first storey and at the shake table level, and the two out-of-plane accelerations at both stories. The list of accelerometers and their location for the impact hammer tests is shown in Table 7.2. Each test consisted of three blows with the hammer with a time lag of a few seconds between the blows. Each record had a total duration of approximately 15 sec and the signals were recorded with a sampling rate of 500 data per second.

Table 7.2. List of the accelerometers used during the impact hammer tests and their location.

<table>
<thead>
<tr>
<th>Location</th>
<th>Instrument</th>
<th>Measurement</th>
<th>Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Accelerometer</td>
<td>Shake Table Acceleration</td>
<td>E-W</td>
</tr>
<tr>
<td>2</td>
<td>Accelerometer</td>
<td>First Storey Acceleration</td>
<td>E-W</td>
</tr>
<tr>
<td>3</td>
<td>Accelerometer</td>
<td>Top Acceleration</td>
<td>E-W</td>
</tr>
<tr>
<td>4</td>
<td>Accelerometer</td>
<td>First Storey Acceleration</td>
<td>N-S</td>
</tr>
<tr>
<td>5</td>
<td>Accelerometer</td>
<td>Top Acceleration</td>
<td>N-S</td>
</tr>
</tbody>
</table>

7.3.2 Choice of Excitation Record for Shake Table Tests

Three different earthquake records were chosen for the shake table tests. The records were selected to provide sufficient excitation for obtaining a non-linear response of the braced frame model at different stages of testing. This is usually achieved by selecting acceleration records with a main frequency of excitation close to the natural frequency of the model. The initial natural frequency of the model was determined from an elastic modal analysis performed using
the computer program SAP 2000, supported by the results from the impact hammer tests. Normalized accelerograms of the three records used during the tests Watsonville, VAN-29 and Joshua Tree are shown in Figure 7.5 while their pseudo acceleration (PSA) response spectra for 5% damping are shown in Figure 7.6.

Figure 7.5. Normalised accelerograms of the records used for shake table tests.
Figure 7.6. Pseudoacceleration response spectra of the records used for the tests.
Watsonville is one of the numerous accelerograms recorded during the well known 1989 Loma Prieta California Earthquake with a magnitude $M = 7.1$. The record was chosen because of its relatively high energy content (Figure 7.6) around one spectrum peak at $T \approx 0.35$ sec. This record can significantly excite the braced frame model, which was expected to be in that period range during some early non-linear stages of the response.

VAN-29 record was one of the accelerograms previously used for the analytical phase of determining the force modification factors for BTF. The strong motion was recorded during the 1971 San Fernando California earthquake with a magnitude $M$ of 6.4. As shown in Figure 7.6, the record has two significant peaks in the spectrum at shorter periods (at 0.12 and 0.19 sec) which coincides with the expected range for the initial natural frequency of the model. The record also has significant amplification potential at periods between 0.32 an 0.4 sec.

The record of the 1992 Landers California earthquake, recorded at the Joshua Tree Fire Station, E-W direction (CSMIP, 1992), already used for the previous set of shake table tests, was chosen as the third record. Based on the initial period estimate, a low to moderate response was expected from the record this time, so it was used in the tests more for comparative purposes than for its ability to induce large non-linear deformations. The earthquake record has a couple of peaks in the spectrum at 0.25 and 0.32 sec, while its highest peak of amplification is at around 0.7 sec, which can induce a large amount of seismic energy to a structure with prolonged period of vibration after some damage had taken place.

During the shake table tests, only motions in the E-W direction of the frame were applied to provide excitation of the braced frame model in the principal direction of stiffness. The shake
Shake Table Tests on a Two Storey Braced Frame Model

Table displacements were controlled by a real time computer controlled system to reflect the acceleration input data from the earthquake record. To maximize the response of the model, the acceleration record of each earthquake record was filtered and factored so that the maximum displacement of the shake table (± 76 mm or 3 inches) was not exceeded. This acceleration level was then referred to as the maximum or 100% level, and all shake table tests were then performed using a maximum input acceleration at some percentage of that level, including the 100 percent one. The maximum levels for the three records chosen were 0.41 g for Joshua, 1.4 g for Watsonville, and 0.45 g for the VAN-29 earthquake record. The high acceleration level that could be obtained from the Watsonville record was also one of the main reasons for choosing that earthquake for the testing program.

7.3.3 Testing Sequence

To investigate the response of the braced frame model at different deformation levels, tests with low, moderate and high amplitude excitations were performed. At the beginning, low intensity tests with each of the three records were carried out to tune the computer program that controls the shake table motion and to determine the interaction of the specimen and the shake table. Low level tests were also needed to provide information regarding the behaviour of the model in the linear range and its initial dynamic characteristics. Moderate intensity tests were useful in studying the non-linear load deformation characteristics of the model at lower deformation levels. Finally, tests at the highest possible acceleration levels for the three records were conducted to investigate the non-linear behaviour of the model at the largest deformation levels possible.
A summary of the most important details about the shake table tests sequence is given in Table 7.3. A total of 16 tests were performed using the three different records. In addition four sets of impact hammer tests were performed after various acceleration levels were reached. As shown in the table, for most of the tests, the maximum recorded shake table acceleration was slightly different than the intended maximum acceleration. Since only one specimen was tested through several different acceleration levels and three different earthquakes, the model response can be used to study the non-linear deformation development, and post moderate earthquake response.

Table 7.3. A summary of shake table tests performed.

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Span % of the Maximum</th>
<th>Acceleration Record</th>
<th>Maximum Acceleration Planned (g)</th>
<th>Maximum Acceleration Recorded (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>IMP-1X</td>
<td>-</td>
<td>Impact In-Plane Direction</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>IMP-1Y</td>
<td>-</td>
<td>Impact Out-of-Plane</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1</td>
<td>10</td>
<td>Joshua Tree</td>
<td>0.041</td>
<td>0.044</td>
</tr>
<tr>
<td>2</td>
<td>10</td>
<td>Watsonville</td>
<td>0.14</td>
<td>0.141</td>
</tr>
<tr>
<td>3</td>
<td>10</td>
<td>VAN-29</td>
<td>0.045</td>
<td>0.061</td>
</tr>
<tr>
<td>4</td>
<td>25</td>
<td>Watsonville</td>
<td>0.35</td>
<td>0.283</td>
</tr>
<tr>
<td>5</td>
<td>25</td>
<td>VAN-29</td>
<td>0.112</td>
<td>0.141</td>
</tr>
<tr>
<td>6</td>
<td>25</td>
<td>Joshua Tree</td>
<td>0.102</td>
<td>0.103</td>
</tr>
<tr>
<td>7</td>
<td>50</td>
<td>VAN-29</td>
<td>0.225</td>
<td>0.275</td>
</tr>
<tr>
<td>8</td>
<td>50</td>
<td>Watsonville</td>
<td>0.7</td>
<td>0.557</td>
</tr>
<tr>
<td>9</td>
<td>50</td>
<td>Joshua Tree</td>
<td>0.205</td>
<td>0.208</td>
</tr>
<tr>
<td>IMP-2X</td>
<td>-</td>
<td>Impact In-Plane Direction</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>IMP-2Y</td>
<td>-</td>
<td>Impact Out-of-Plane</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>10</td>
<td>75</td>
<td>VAN-29</td>
<td>0.337</td>
<td>0.383</td>
</tr>
<tr>
<td>11</td>
<td>75</td>
<td>Watsonville</td>
<td>1.05</td>
<td>0.871</td>
</tr>
<tr>
<td>12</td>
<td>100</td>
<td>Watsonville</td>
<td>1.4</td>
<td>1.192</td>
</tr>
<tr>
<td>13</td>
<td>100</td>
<td>VAN-29</td>
<td>0.45</td>
<td>0.478</td>
</tr>
<tr>
<td>IMP-3X</td>
<td>-</td>
<td>Impact In-Plane Direction</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>IMP-3Y</td>
<td>-</td>
<td>Impact Out-of-Plane</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>14</td>
<td>100</td>
<td>Watsonville</td>
<td>1.4</td>
<td>1.101</td>
</tr>
<tr>
<td>15</td>
<td>100</td>
<td>VAN-29</td>
<td>0.45</td>
<td>0.507</td>
</tr>
<tr>
<td>16</td>
<td>100</td>
<td>Joshua Tree</td>
<td>0.41</td>
<td>0.404</td>
</tr>
<tr>
<td>IMP-4X</td>
<td>-</td>
<td>Impact In-Plane Direction</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>IMP-4Y</td>
<td>-</td>
<td>Impact Out-of-Plane</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
7.4 RESULTS AND DISCUSSION

7.4.1 Impact Hammer Tests

The impact hammer tests were conducted before the beginning of the tests and in between the tests as shown in Table 7.3. Each impact test consisted of three blows with the hammer with a few seconds time lag between the blows. Typical acceleration signals recorded at the top of the model during the in-plane and out-of-plane impact tests 3X and 3Y are shown in Figure 7.7.

Figure 7.7. Acceleration signals at the top of the model during 3X and 3Y impact tests.
The fundamental natural frequency of the model during each test was determined from the peak of the spectrum obtained from the Fast Fourier Transform (FFT) of the signal recorded on the top of the model. For example, the FFT functions obtained from the accelerations of the tests 3X and 3Y are shown in Figure 7.8. The fundamental frequencies obtained during all the tests in both directions are presented in Table 7.4.

Figure 7.8. Spectral amplitude of the top accelerations from impact tests 3X and 3Y.

The impact hammer tests were also used to determine the damping of the fundamental mode of model at various stages. The damping was determined from the free vibration decay of
amplitudes of horizontal acceleration measured at the top of the model. The damping values found for the first mode of vibration are also shown in Table 7.4.

Table 7.4. Frequency and damping values for the fundamental mode of the model.

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Frequency (Hz)</th>
<th>Period (sec)</th>
<th>Damping (% of critical)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-X</td>
<td>8.50</td>
<td>0.117</td>
<td>1.9</td>
</tr>
<tr>
<td>1-Y</td>
<td>3.81</td>
<td>0.262</td>
<td>2.0</td>
</tr>
<tr>
<td>2-X</td>
<td>8.07</td>
<td>0.124</td>
<td>2.1</td>
</tr>
<tr>
<td>2-Y</td>
<td>3.77</td>
<td>0.265</td>
<td>2.3</td>
</tr>
<tr>
<td>3-X</td>
<td>7.54</td>
<td>0.133</td>
<td>2.2</td>
</tr>
<tr>
<td>3-Y</td>
<td>3.74</td>
<td>0.267</td>
<td>2.7</td>
</tr>
<tr>
<td>4-X</td>
<td>6.7</td>
<td>0.149</td>
<td>3.7</td>
</tr>
<tr>
<td>4-Y</td>
<td>3.76</td>
<td>0.266</td>
<td>3.9</td>
</tr>
</tbody>
</table>

As shown in the table, the braced frame model was considerably stiffer in the X (in-plane) direction than in the Y (out-of-plane) direction. This was expected since the steel angle cross braces were much more slender than the in-plane glulam braces. The fundamental frequency in the X direction changed from 8.5 Hz before the tests to 6.7 Hz after the last test, reflecting the reduced stiffness of the frame after severe shaking has taken place. These results, however, should be viewed with caution because the intensity of the impact is very low and the forces generated are negligible compared to the inertia forces during the shake table tests. The tests therefore cannot be expected to simulate the real dynamic behaviour of the model. It is also well known that the impact hammer tests cannot accurately simulate the properties of the model after significant deformations or damage has occurred. Although not exactly precise, and almost always higher than the real values, the frequencies obtained with the impact hammer tests can be used as an illustration of the gradual stiffness degradation of the model caused by the progressive development of damage during the tests.
During the four impact tests in the X direction the damping increased from 1.9 % at the beginning to 3.7 % at the end. This suggests that the assumed damping value of 3% used in the analyses presented in the previous chapter was wisely chosen. In the out-of-plane direction no significant change in frequency (stiffness) was observed during the four tests. An increase in damping was found, however, from 2.0% at the beginning to 3.9% at the end. The differences in initial fundamental frequencies in the X and Y directions were relatively significant so that strong cross-coupling effect such as torsion or beating were not expected.

7.4.2 Shake Table Tests

7.4.2.1 Data Pre-Processing

The most important results from shake table test are the time histories recorded from the different sensors placed on the model. Before the data recorded from the sensors placed on the model can be used for quantification of the dynamic response, a detailed analysis of the signals in the time and frequency domain (pre-processing) must be performed. The pre-processing was performed using a spreadsheet developed for that purpose in the commercial mathematical software package Mathcad. A printout of one version of the spreadsheet is given in Appendix A of the thesis.

At the beginning, different operations were performed to transform the recorded voltages to acceleration and displacement measurements. Then the signals were windowed and averaged, based on the values of the initial voltage (quantity) associated with the zero reading of a given sensor. At this point, the signals still contained some unwanted influences that did not relate to the response of the model, usually referred to as noise. A high frequency noise is typically the
result of an imperfection in the data acquisition system or in the instrumentation network. This was eliminated by using Butterfly type four pole low pass filters, with a cut-off frequency of 15 Hz. A low frequency noise in the signals can be triggered by the sensors themselves if not properly balanced or if not adequately mounted. This noise was eliminated by using two-pole Butterfly type filters with a cut-off frequency of 0.2 Hz.

Finally, the records from all sensors were plotted for all tests along with their Fourier Spectra. Records from all sensors from one test were compared and based on the registrations from different sensors, the exact frequency range of the response could be obtained. In addition, a comparison of the records of a particular sensor during all the tests was performed. Crosschecking of records from a small number of tests is not sufficient because good registration can be obtained during one test but then sensor damage or some other fault can result in a bad record during the next test. Generally, all manipulations with the basic record should be done very carefully with good planning and well chosen filters. If not done so, significant information may be lost such as the influence of the higher modes or the presence of residual deformations, so a skewed impression of the response of the model can be obtained.

7.4.2.2 Acceleration Time Histories

Acceleration records from the sensors placed along the height of the model provide very useful information about the model behaviour during the tests and are a direct indication of force levels. Figures 7.9a and 7.9b show the accelerations recorded at the top of the model from five different tests with the VAN29 record as base excitation at different acceleration levels. In the tests number 3, 5 and 7 (up to 0.275 g) the amplitudes of the response grew proportionally to the
intensity of the input acceleration. In the next test (run 10), stagnation in the response was observed despite almost 40% increase in the input acceleration and a slight change in the acceleration record shape took place. During run 13 (100% VAN29) a radical change of the shape of the acceleration record occurs. Finally, test number 15, although at approximately the same excitation level as the previous one, showed significant differences in the acceleration record with a 40% reduction in peak acceleration.

Figure 7.9a. Accelerations at the top of the model for various levels of VAN29 input.
Figure 7.9b. Accelerations at the top of the model for various levels of VAN29 input.

Figure 7.10a. Accelerations at the top of the model for various levels of Watsonville input.
Figure 7.10b. Accelerations at the top of the model for various levels of Watsonville input.

Figure 7.11a. Accelerations at the top of the model for various levels of Joshua Tree input.
Even from a cursory analysis of the time histories, it is evident that the system remained linear elastic during the early tests. Some slight changes had occurred in the model stiffness and damping characteristics during test number 10, suggesting a non-linear deformations. More significant changes were observed during tests 13 and 15 indicating more significant wood crushing in connections and reduced stiffness. Similarly to Figure 7.9, Figures 7.10 and 7.11 show the accelerations recorded at the top of the specimen during different tests with the Watsonville and Joshua Tree records as base excitation. Similar conclusions as those mentioned above can be reached for the tests with these two records.
Figure 7.12. Accelerations along the height of the model for test 7 – VAN 29 (50%).

Figure 7.13. Accelerations along the height of the model for test 13 – VAN 29 (100%).
Figures 7.12, 7.13 and 7.14 show the accelerations along the height of the model (at the bottom, first storey and second storey) for tests number 7, 13 and 15, all using the VAN-29 earthquake input record. By comparing the accelerograms from tests 13 and 15 (both 100% span), the difference in the structural characteristics is evident due to non-linear deformations exhibited during test 13. For test 13 (100% VAN-29 first time) the maximum acceleration at the top was almost 2.9 times the input acceleration, while in test 15 the amplification was reduced to around twice the input acceleration. The maximum accelerations recorded at the shake table level, first storey and the second storey of the model, as well as the dynamic amplification factors are given in Table 7.5. The dynamic amplification factors were calculated as the ratio between the maximum top acceleration versus the maximum input acceleration.
Table 7.5. Maximum accelerations recorded at each storey of the model.

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Earthquake Record and Span</th>
<th>Shake Table Acceleration (g)</th>
<th>First Storey Acceleration (g)</th>
<th>Second Storey Acceleration (g)</th>
<th>Dynamic Amplification</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Joshua, 10%</td>
<td>0.044</td>
<td>0.043</td>
<td>0.057</td>
<td>1.295</td>
</tr>
<tr>
<td>2</td>
<td>Watsonville, 10%</td>
<td>0.111</td>
<td>0.117</td>
<td>0.162</td>
<td>1.459</td>
</tr>
<tr>
<td>3</td>
<td>VAN-29, 10%</td>
<td>0.061</td>
<td>0.094</td>
<td>0.136</td>
<td>2.229</td>
</tr>
<tr>
<td>4</td>
<td>Watsonville, 25%</td>
<td>0.283</td>
<td>0.167</td>
<td>0.392</td>
<td>1.385</td>
</tr>
<tr>
<td>5</td>
<td>VAN-29, 25%</td>
<td>0.141</td>
<td>0.251</td>
<td>0.342</td>
<td>2.425</td>
</tr>
<tr>
<td>6</td>
<td>Joshua, 25%</td>
<td>0.103</td>
<td>0.121</td>
<td>0.150</td>
<td>1.456</td>
</tr>
<tr>
<td>7</td>
<td>VAN-29, 50%</td>
<td>0.275</td>
<td>0.373</td>
<td>0.581</td>
<td>2.112</td>
</tr>
<tr>
<td>8</td>
<td>Watsonville, 50%</td>
<td>0.557</td>
<td>0.682</td>
<td>0.961</td>
<td>1.725</td>
</tr>
<tr>
<td>9</td>
<td>Joshua, 50%</td>
<td>0.208</td>
<td>0.24</td>
<td>0.263</td>
<td>1.264</td>
</tr>
<tr>
<td>10</td>
<td>VAN-29, 75%</td>
<td>0.383</td>
<td>*</td>
<td>0.625</td>
<td>1.632</td>
</tr>
<tr>
<td>11</td>
<td>Watsonville, 75%</td>
<td>0.871</td>
<td>0.926</td>
<td>1.597</td>
<td>1.833</td>
</tr>
<tr>
<td>12</td>
<td>Watsonville, 100%</td>
<td>1.192</td>
<td>1.412</td>
<td>2.245</td>
<td>1.883</td>
</tr>
<tr>
<td>13</td>
<td>VAN-29, 100%</td>
<td>0.478</td>
<td>0.815</td>
<td>1.379</td>
<td>2.885</td>
</tr>
<tr>
<td>14</td>
<td>Watsonville, 100%</td>
<td>1.10</td>
<td>0.980</td>
<td>1.885</td>
<td>1.713</td>
</tr>
<tr>
<td>15</td>
<td>VAN-29, 100%</td>
<td>0.507</td>
<td>0.379</td>
<td>1.039</td>
<td>2.049</td>
</tr>
<tr>
<td>16</td>
<td>Joshua, 100%</td>
<td>0.404</td>
<td>*</td>
<td>1.008</td>
<td>2.495</td>
</tr>
</tbody>
</table>

* Not reliable record because of pounding effects between the steel mass blocks.

7.4.2.3 Displacements and Storey Drifts

Along with accelerations, the frame displacements and storey drifts are the most important data from the shake table tests, and are regarded as the basic parameters of the global structural response. The displacements represent the response of the model along its “global coordinates”, and are very often used for comparison of the experimental response with the analytical results from mathematical models. Time histories of the relative displacement at the top of the north frame (Ch. 14) for different tests with the VAN-29 record are shown in Figure 7.15, while similar displacement records from tests with the Watsonville earthquake are shown in Figure 7.16.
Figure 7.15. Displacements at the top of the north frame for various levels of VAN-29 input.

Figure 7.16a. Displacements at the top of north frame for various levels of Watsonville input.
Displacement records shown in Figures 7.15 and 7.16 show a steady increase in the maximum displacement levels corresponding to the increase in the input acceleration. For the VAN-29 record, for instance, at lower acceleration levels an increase in maximum base acceleration of 2.5 times (from 10% to 25% of maximum) resulted in a three-fold increase of maximum displacement (from 1 mm to 3 mm). At medium to high acceleration levels (from 50% to 75% of maximum) the differences were found to be greater. A 50% increase in acceleration resulted in a displacement increase of 4.15 times (from 5.6 mm to 23.4 mm). For test number 15, which had the maximum acceleration of VAN-29 for the second time (post-disaster earthquake), an increase of maximum displacement from 53.7 mm to 62.8 mm was found. Similar findings were noticed for the other two input records.
During the first half of the testing program, the Watsonville and VAN-29 records produced higher displacement levels than the Joshua Tree record at same percentages of the maximum input acceleration. For that reason, the Joshua Tree record was not used for 75 % and 100 % span tests. This was expected because the Joshua earthquake did not have a significant energy content in the frequency range associated with the model characteristics in the early shaking. The Joshua record though, was used for the last test (run 16) after the model had experienced significant non-linear deformations due to previous records. This time a significant response was experienced and the time history of the relative displacement obtained at the top of the north frame from this test is shown in Figure 7.17.

The maximum relative displacements at the top of the north frame were approximately 62 mm for the three earthquakes at 100% span, namely 62.3 mm for Watsonville, 62.8 mm for VAN29 and 62.7 mm for Joshua Tree. The displacements at the top of the south frame were found a little smaller than the corresponding ones on the north frame. The difference was in the range from 10 % at lower acceleration levels to a maximum of 23 % at the highest acceleration levels. The difference in displacements obtained from the north and the south frame induced some torsional motions on the model. Figure 7.18 shows the time histories of the torsional components at the top of the model for four different shake table tests.
Storey drifts are important in that they are directly related to the deformations and forces generated in the braces. Figures 7.19 and 7.20 show the drifts of both stories measured on the north frame during tests 14 and 15. As expected, the drifts were slightly higher in the first storey most of the time during the response, with certain peaks being up to twice as large as those of the second storey. This suggests that higher brace deformations are expected in the lower braces of the model. Again, the south braced frame experienced lower drifts on both storeys compared to the north frame. For more precise location of the deformations in each storey, the deformations in the braces should be analysed.
Figure 7.19. Storey drifts recorded during test 14 north frame – Watsonville 100%.

Figure 7.20. Storey drifts recorded during test 15 north frame– VAN-29 100%.

7.4.2.4 Brace Deformations

The deformations in the braces are directly related to the storey drifts obtained during the tests. The lower storey drifts in the south frame resulted in smaller brace deformations for this frame as well. In addition, smaller brace deformations were experienced in the upper braces than in the
lower braces of both the north and south frames. Figures 7.21 to 7.24 show the time histories of the deformations in the upper and lower braces for both frames for the shake table tests 11 to 16.

As shown in the figures, the deformation levels of upper and lower brace of the north frame were about 30 to 50% higher than the corresponding deformations of the braces of the south frame, depending on the earthquake and the intensity level. The deformations of the lower braces were from two to 2.6 times higher than the deformations in the upper braces for both frames, indicating that the bulk of the non-linear deformations can be expected in the braces of the first storey.

As expected, the deformations in the braces were not equally distributed between the two connections of the brace. An analysis on the deformation records of the upper and lower connections of both braces on the south frame showed that difference. For all the tests, the upper connection of the top brace experienced higher deformations than the lower one, while the bottom connection of the first storey brace showed more deformations than the upper one. Time histories of the connection deformations on the upper and the lower brace of the south frame for test 14 are shown in Figures 7.25 and 7.26 respectively.
Figure 7.21. Deformations of the upper brace of the north frame during the last six tests.
Figure 7.22. Deformations of the lower brace of the north frame during the last six tests.
Figure 7.23. Deformations of the upper brace of the south frame during the last six tests.
Figure 7.24. Deformations of the lower brace of the south frame during the last six tests.
Figure 7.25. Deformations in both connections of the upper brace of the south frame - test 14.

Figure 7.26. Deformations in both connections of the lower brace of the south frame - test 14.
7.4.2.5 Load Deformation Relationships

From the data presented so far, a series of conclusions can be drawn on the seismic behaviour of the braced frame specimen. For the completeness of the picture, it is necessary to present some relations between the different parameters of the structural response. One such relation is the load-deformation relationship of the braces, the elements that underwent the non-linear deformations during the tests. Figures 7.27 to 7.31 show the load deformation characteristics of both the top and bottom braces for each of the two braced frames obtained during the different shake table tests. The deformations of the braces were measured directly from the tests as shown previously. The forces in the braces were calculated by multiplying the masses on each level with the accelerations measured at corresponding levels, while taking into account the brace angle at each time step of the test.

Generally, the 1 to 10, due to lower acceleration levels produced much lower deformations (and forces) than tests 11 to 16. Therefore, the load-deformation characteristics form the early tests are not presented, except the relationships of the lower brace of the north frame for tests 5 to 10 (Figure 7.27). The relationships of the other braces closely follow the shape of the results presented in Figure 7.27 for tests 5 to 10, so were not presented here. Figures 7.28 to 7.31, the load-deformation characteristics are presented for all braces during tests 11 to 16, where more significant deformations in the braces had occurred.
Figure 7.27. Load-deformation relationships of the bottom brace - north frame.
Figure 7.28. Load-deformation relationships of the top brace - north frame.
Figure 7.29. Load-deformation relationships of the bottom brace - north frame.
Figure 7.30. Load-deformation relationships of the top brace - south frame.
Figure 7.31. Load-deformation relationships of the bottom brace - south frame.
During the early low level tests, the behaviour of the braces and the braced frame model in general was linear elastic. Somewhat higher deformation levels were experienced in the tests at 50% span of the earthquakes (tests 7 and 9), but still the behaviour of the braces was linear elastic. Test 8 due to a higher acceleration level (0.56g), was the first to show some non-linear deformations in the braces (Figure 7.27), with small and thin hysteresis loops characterising the behaviour.

The hysteresis loops obtained for later tests were typical of those for members with riveted connections. The same pinching of the hysteresis curves occurred as found during the cyclic tests. This was a result of the irrecoverable crushing of the wood that leaves a gap at load reversal. During subsequent excursions through this gap region, the lateral resistance and energy dissipation almost entirely occurred in the rivet connectors. The hysteresis loops were found to have a higher pinching effect as the tests progressed and higher deformation levels were reached. As mentioned before, the deformations in the braces of the north frame were higher than those of the corresponding braces of the south frame. Understandably, the lower braces experienced higher deformation and load levels in both frames. The top brace of the north frame had a maximum deformation of about 6.6 mm while the bottom one had a maximum deformation of 6.12 mm. The maximum axial forces generated in the lower brace were, however, only 10 to 14% higher than those obtained in the upper braces.

The last three shake table tests (14, 15 and 16) showed reduced loads and stiffness for same acceleration levels experienced in previous tests. This strength and stiffness deterioration is characteristic for timber structures when experiencing deformation levels less than previously
reached. The reduction of strength and stiffness was more progressive and most significant in tests 15 and 16 which represent the case of post disaster earthquakes.

### 7.4.2.6 Frequency content of the response

By analysing the frequency content of the response, valuable information can be gathered about the dynamic effects of the input motion on the tested model. When the dynamic testing is conducted in a series of tests with increasing amplitude as it was done here, the frequency content of the input motion does not change for a certain earthquake record. Since the input frequency content does not change, the changes in the frequency content of the response are only due to changes in the structural properties. The changes of the dynamic effects in frequency domain give information on the changes in the stiffness properties of the model only and are not affected by the changes of the damping capacity of the model.

A frequency domain analysis can also deliver approximate values of the frequency of vibration of the model. It should be mentioned that the exact value of the frequency can not be obtained because all the frequency components are not equally represented in the input motion for the particular range of analysis. The analysis can only be done by comparing the spectral amplitudes of the input motion and the response and based on that, an estimate can be made of the properties of the model. To determine the exact values of the frequency, a testing method that covers the entire frequency range such as forced or ambient vibration technique can be used.

The Fourier amplitude spectra of the three input motions used during the tests and the spectra of top accelerations obtained at the corresponding tests 1, 2 and 3 are shown in Figures 7.32a and
7.32b, respectively. These three tests at 10% span were used to characterise the linear behaviour of the model at the early stage.

Figure 7.32. a) Fourier amplitude spectra of the three input motions used during the tests and b) the spectra of top accelerations obtained during the corresponding tests.
As shown in the figure, the spectra obtained at the top of the model are different at certain frequency ranges when compared to those of the corresponding input accelerations. The most recognisable difference in all three spectra is the appearance of significant amplification in the frequency range of 7 to 9 Hz, suggesting that the natural period of the model during those tests is in that particular frequency range. By defining the frequency of the maximum spectral amplification as the ratio of the amplitudes of the input and the output spectrum, the frequency of vibration of the model during each test can be more closely determined. Figure 7.33 shows the amplitude ratio of the input and the output spectrum for test number 1 that uses a Joshua Earthquake record as the input motion. The other two ratios are very similar to the one presented in Figure 7.33 and for that reason are not presented here.

Figure 7.33. The amplitude ratio of input and output spectrum for test 1 - Joshua Earthquake.

The maximum amplification in the case of the Joshua record was found to be at a frequency of 8.71 Hz, while in the other two cases the frequencies were 8.35 Hz for the Watsonville earthquake and 8.64 Hz for the VAN-29 record. The average of these three tests yields a frequency of 8.56 Hz, which is almost identical to the experimentally determined value of 8.5 Hz. As mentioned before, although the both values are very close, they should be taken as the
approximate value for the frequency of the model. Even so, the obtained frequencies show that the impact hammer technique can give reliable data on the frequency of a model in the early stages of the testing. This method, however, due to the reasons mentioned before, does not yield the accurate frequencies of the model when larger deformations or damage has occurred.

Figure 7.34. Fourier amplitude spectra for six tests from 10% to 100% - Watsonville earthquake.
The changes in the properties of the model during the tests can be obtained by comparing the spectra from the top acceleration for different tests with same input earthquake. For instance, Figure 7.34 shows the amplitude spectra obtained from six tests from 10% to 100% span which used the Watsonville earthquake as an input. As mentioned before, the frequency of vibration was around 8.5 Hz during test number 2. During test number 4 a change in the spectrum can be noticed in the range between 5.5 Hz to 7 Hz, suggesting that the frequency of vibration of the model can be in that range. Later, the spectrum of test number 8 shows amplification in the range between 5 and 6.5 Hz. while the peaks in the frequency range higher than 8 Hz diminish. This frequency obtained for the model from the experimental tests after test number 8 (8.07 Hz) is different and out of this range. As expected, the impact hammer tests yielded a higher frequency after significant deformations had occurred in the model. This is mostly because the energy from the hammer test is not sufficient to excite all the dynamic motions otherwise experienced during the dynamic test.

The difference between the experimental values and the values estimated based on the spectra increased as the testing progressed. Test number 11 showed an amplification of the peaks between 4 and 5 Hz, and reduction of the difference with the peaks between 5 and 6 Hz. The frequency at this point is estimated to be in the range of 4 and 6 Hz, with the exact value probably being closer to the 4 Hz value. At this point we can say for certain that the frequency of the model is not higher than 6.5 Hz because the peaks in this range are well diminished with respect to the others. Finally, in test 14 the changes could be noticed around the absolute maximum peak of the spectrum between 2 and 3 Hz, indicating that the frequency of the model is probably between these values. The reduction of the frequency of the model in each
consecutive test represents a change in its stiffness properties due to deformations and wood crushing occurred in the brace connections.

7.4.2.7 Vibration Shapes and Damage Development

Through careful analysis of the displacement records and storey drifts throughout the testing it was noticed that for all three different seismic inputs and all different intensities, the model vibrated in its fundamental mode. Knowing that for all input earthquakes the predominant frequencies die down after 9 Hz and the initial fundamental frequency of the model being 8.5 Hz, any influence of the second mode of vibration was not expected in the response. Figure 7.35 shows the normalised coordinates of vibration of the model subjected to the three different earthquake motions at the time of maximum deformations.

![Figure 7.35. Shapes of the vibration of the model for different earthquakes.](image)

By closely analysing the displacement records it could be noticed that with increasing the earthquake intensity, the participation of the first storey drift in the total deformation of the model increases. This is mostly due to concentration of the non-linear deformations in the
riveted connections of the lower brace and presence of wood crushing and gap formation in those connections. During the shaking the structure behaved as designed using the capacity design approach. All beam to column connections throughout the frame model remained intact while all the deformations were concentrated in the brace connections. As shown previously, the maximum deformations were observed in the bottom connections of the lower brace, especially on the north side. The maximum connection deformations were in the range of ± 6 mm which in our judgement just fell short of reaching the maximum load in the connections. In addition, no characteristic rivet failure modes could be observed even though the mentioned connections underwent relatively high deformation levels.

7.5 PUSH-OVER TESTS

Because the failure of the model was not reached at the end of the shake table tests, it was decided to conduct a push-over test in order to determine the capacity of the structure. The push-over test was performed while the model was still fixed to the shake table.

7.5.1 Test Setup and Instrumentation

A picture of the model ready for conducting a push-over test is shown in Figure 7.36, while the 3-D drawing of the same model is shown in Figure 7.37. The masses were taken down after finishing the shake table tests. A hollow structural section (HSS 127 x 76 x 6.4 mm) beam was placed across the model at the top of the east side. The beam was bolted on both sides to each of the two frames of the model. Another longer arm beam with the same cross section was attached at right angle to connect the crossbeam to a supporting column.
Figure 7.37. A picture of the braced frame model ready for the push-over test.

Figure 7.38. 3-D drawing of the braced frame model ready for the push-over test.
The connections on both sides of the connecting link beam were pinned allowing rotations at both ends. A load cell at the end of the link beam was used to measure the load generated at the top of the model. Because the location of the supporting column was not exactly at the center of the braced frame model, the link beam (load cell) was not connected at the mid span of the cross beam. The connection point was approximately one third of the crossbeam span away from the south frame.

During the pushover test the model was fixed to the shake table. Using the hydraulic controls, the shake table was placed as far as possible to the west, away from the supporting column, so the maximum available stroke of the shake table (150 mm) could be used. The link beam was then extended to the length needed so it could be attached to the cross beam without any slack. The push-over test was then conducted by moving the shake table (and the bottom of the model) towards the east side, while keeping the top of the model in place with the link beam. The speed of moving the table was 1.058 mm/sec (24 sec/inch). After the entire stroke of the shake table was exhausted, the model was pushed back using the same speed until zero displacement was reached. A total of 10 different sensors were installed to monitor the behaviour of the braced frame model during the push-over test. The location of the instruments used to record the signals on both frames was the same as for the shake table tests (shown in Figures 7.3 and 7.4), with the difference that acceleration and displacement sensors were not used. The information was recorded using one of the existing data acquisition systems in the laboratory (Labview), with a sampling rate of 50 data points per second. A list of the sensors and the measured quantities are given in Table 7.6. The deformation of the support column was not expected to be significant to affect the results so for that reason was not measured.
Table 7.6. List of the sensors used during the push-over test and their location.

<table>
<thead>
<tr>
<th>Location</th>
<th>Instrument</th>
<th>Measurement</th>
<th>Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Load Cell</td>
<td>Generated Load at the Top</td>
<td>Horizontal</td>
</tr>
<tr>
<td>2</td>
<td>Displacement Sensor</td>
<td>Second Storey, Entire North Brace</td>
<td>Diagonal</td>
</tr>
<tr>
<td>3</td>
<td>Displacement Sensor</td>
<td>First Storey, Entire North Brace</td>
<td>Diagonal</td>
</tr>
<tr>
<td>4</td>
<td>Displacement Sensor</td>
<td>Second Storey, Entire South Brace</td>
<td>Diagonal</td>
</tr>
<tr>
<td>5</td>
<td>Displacement Sensor</td>
<td>First Storey, Entire South Brace</td>
<td>Diagonal</td>
</tr>
<tr>
<td>6</td>
<td>DCDT</td>
<td>Top Connection II Storey South</td>
<td>Diagonal</td>
</tr>
<tr>
<td>7</td>
<td>DCDT</td>
<td>Bottom Connection II Storey South</td>
<td>Diagonal</td>
</tr>
<tr>
<td>8</td>
<td>DCDT</td>
<td>Top Connection I Storey South</td>
<td>Diagonal</td>
</tr>
<tr>
<td>9</td>
<td>DCDT</td>
<td>Bottom Connection I Storey South</td>
<td>Diagonal</td>
</tr>
<tr>
<td>10</td>
<td>Displacement Sensor</td>
<td>Shake Table Displacement E-W</td>
<td>Horizontal</td>
</tr>
</tbody>
</table>

7.5.2 Results and Discussion

Because the top of the model was not held with the link beam at center point, a significant difference in deformation of the south and the north frame of the model was experienced. That was expected since the point of holding was much closer to the south frame, so a larger component of the load was transferred to this frame. The deformation of the braced frame model when maximum deformation is reached is shown on the picture presented in Figure 7.39. As shown on the picture, the difference in deformations of the south and north frame is clearly visible resulting in a torsional deformation shape for the frame.
The load-deformation relationship for the entire frame based on the measured load and the shake table deformation is shown in Figure 7.40. The maximum load obtained was 73.3 kN at deformation level of 68.8 mm. From that point on, the load started to drop in almost a linear relationship as the deformation increased and reached a value of 45.3 kN when the maximum deformation of 126.5 mm was reached. The load-deformation behaviour obtained was a characteristic one for a ductile structure able to sustain large deformations without sudden and significant drop in deformations.
As mentioned above, the participation of the south frame was much greater in the overall frame behaviour than that of north frame. The differences in brace deformations can be seen through the given load-deformation relationship of the braces in the south and the north frame as shown in Figure 7.41. The obtained maximum load in the braces of the south frame was 54.26 kN which is about 13% higher than the maximum load obtained in the braces during the shake table tests. The maximum brace deformation was experienced in the bottom brace of the south frame at 70.6 mm, while the top brace had 56.3 mm elongation. These values were significantly higher than the corresponding deformations of the braces of the north frame (13.3 mm and 5.5 mm, respectively). The latter deformation levels (in the braces of the north frame) were even less than the deformation levels experienced during the shake table tests. Because of that, the curves in Figure 7.41 for the north braces show a typical behaviour of a connection (or element) that undergoes deformation in the already formed gap due to previous wood crushing.
During the test it was also noticed that deformations were not equally shared between the connections in a certain brace. Maximum deformations in the south frame were experienced in the bottom connection of the lower brace (52.7 mm), followed by the top connection of the upper brace (33.3 mm), while the other two connections experienced much lower deformations (around 5 mm each). The failure mode of the riveted connections consisted of significant yielding of the rivets during the first half cycle until the maximum deformation was reached. This was followed by an almost complete pullout of the rivets during the reversal half cycle. The bottom riveted connection of the lower brace of the south frame after the test is shown in Figure 7.42. The picture shows that the connection experienced a typical failure mode of a riveted connection as described above.
SUMMARY

In this chapter results from a series of shake table tests and a pushover test on a two-storey braced timber frame model with riveted connections are presented. A total of 16 shake table tests were performed using three different earthquakes as input motions and different acceleration levels. The results showed that the model vibrated in its fundamental mode during all the tests and was able to sustain the highest possible acceleration of 1.1g without failure. The load-deformation characteristics of the braces obtained during the tests showed hysteresis loops typical for riveted connections. The two braced frames of the model did not experienced the same deformation levels during the tests. In addition, the connections in each of the braces of
the same frame did not experience the same deformation either. Because the failure state of the model was not reached during the shake table tests, a pushover test was conducted. The tests revealed the declining part of the load-deformation relationship for the model, which was characterised by significant ductility and large deformations.
8. ANALYTICAL PREDICTION OF THE MODEL RESPONSE

The results from a series of shake table tests on a two storey braced frame model were presented in the previous chapter. A complete picture of the frame response was presented including the non-linear load-deformation relationships of the braces with riveted connections. The main objective of the research described in this chapter is to analytically predict the non-linear behaviour of the model obtained during the shake table tests. Similarly to the other analytical predictions presented in the thesis, the response was predicted with an analytical model developed in DRAIN 2DX, using the already described “Florence Model” for modelling the non-linear connection behaviour. By comparing the results obtained from the analytical prediction and the shake table tests, the efficiency of the analytical model for prediction could be determined. A satisfactory analytical prediction of the non-linear behaviour would further strengthen the results from the analytical studies presented in the previous chapters.

8.1 ANALYTICAL MODELS

For determining the initial dynamic properties and obtaining the behaviour of the braced frame specimen under different earthquakes, two types of analytical models were developed: 3-D linear elastic model and 2-D (planar) non-linear model. While the linear model can represent well the three-dimensional nature of the tested frame, it cannot predict the non-linear hysteretic behaviour of the wood brace members as can the non-linear two-dimensional model.
8.1.1 3-D Linear Elastic Model

The 3-D linear elastic model was developed using the computer program SAP90 (Wilson and Habibullah, 1992). Among other features, the program can perform modal analysis, linear elastic static and time history dynamic analysis. The three-dimensional linear elastic finite element model is presented in Figure 8.1. Prior to the shake table tests, the model was mainly used to predict the initial dynamic properties of the braced frame and determine the approximate level of forces generated in the elements due to various earthquake motions. After the tests, the model was used for linear dynamic time history analyses for preliminary purposes.

![Figure 8.1. A 3-D linear elastic model of braced timber frame (SAP90).](image)

All elements were modelled as linear elastic frame type elements with their centre-to-centre length, cross sectional and material properties. Material properties of glulam members (thicker lines in Figure 8.1) were modelled according to the properties obtained from material tests.
presented in chapter 4. The masses were concentrated at four nodes of the model at both storey levels. The lumped mass values at the nodes included the mass of the concrete or steel blocks as well as the mass of all glulam or steel members of the frame. To account for the in-plane rigidity of the concrete and steel mass blocks at both storey levels, a global constraint option was used in the program to constrain (slave) the displacements of all other nodes of a storey to the displacement of one chosen storey node. The base supports of the frame allowed free rotation in the in-plane longitudinal direction, while all other rotations were constrained. The glulam braces were also allowed free in-plane rotations at both ends. For the linear dynamic analyses, the damping for the model was chosen to be the same as the value determined from the impact hammer tests.

8.1.2 2-D Non-Linear Analytical Model

Since the computer program DRAIN-2DX supports only a non-linear dynamic analysis of planar structures, a two dimensional model was developed to represent the 3-D braced frame model from the shake table tests (Figure 8.2). Because of the symmetry of the original braced frame specimen, only one frame was modelled in the 2-D model. The non-linearity in the model was essentially concentrated in the connections of the glulam braces, while all other elements (beams and columns) were modelled to be linear elastic. As mentioned before, the brace behaviour was modelled using a subroutine developed at the University of Florence in Italy (Ceccotti and Vignoli, 1989). The model reproduces the path of a typical load-deformation hysteresis loop, keeping track of the maximum previous deformation, and using different slopes to take into account the connection properties associated with wood crushing. All elements were
allowed to have free rotations in the plane of the frame at both ends. The frame base supports
also allowed free in-plane rotation, representing the bottom hinges of the braced frame model.

The masses were concentrated at nodes 3 to 6 representing the weight of the concrete and steel
blocks respectively. The non-linear behaviour of the braces was modelled directly from the
hysteresis loops obtained from the tests, since the loops incorporated the non-linear behaviour of
both connections plus the linear elastic behaviour of the glulam brace. The viscous damping
coefficients included in the analysis were calculated using the damping ratios and the natural
periods of the frame obtained from impact hammer tests. For large deformations, the hysteretic
damping is automatically included in the analysis by the load-deformation curve for the brace,
so the relative contribution of viscous damping is diminished. For small deformations, however,
the skeleton model stays in the elastic range so the model cannot include the hysteretic damping
found in reality. Therefore, the inclusion of sufficient viscous damping to simulate low
amplitude energy dissipation was important.
8.2 RESULTS AND DISCUSSION

8.2.1 Modal Analysis

To obtain the natural frequencies of the models and to compare these values with the measured ones, a modal analysis was performed using both the three dimensional linear elastic model and the two dimensional non-linear model. In addition, the modal analysis was used as a tool to calibrate the analytical models.

![First two mode shapes obtained from the linear elastic 3-D model (SAP90).](image)

The first two mode shapes obtained from the modal analysis of the linear 3-D model are shown in Figure 8.3. The first mode \((T = 0.138 \text{ sec})\) was found to be in the out-of-plane \(Y-Y\) direction, which confirms the experimental findings. Its importance, however, is not significant because the earthquake motion is in the \(X-X\) direction, where the second mode was detected with a period of \(T=0.116 \text{ sec}\). This value is exactly the same as the experimentally determined one. The
second mode in the X-X direction had a very high frequency so its influence on the total response of the model would be insignificant. The participation of the first two modes in their directions with respect to the total response was found to be 95.4 % and 94.3 % respectively.

Table 8.1. Initial dynamic characteristics of both analytical models.

<table>
<thead>
<tr>
<th>Model</th>
<th>Mode</th>
<th>Direction</th>
<th>Frequency (Hz)</th>
<th>Period (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3-D</td>
<td>I</td>
<td>Y-Y</td>
<td>7.24</td>
<td>0.138</td>
</tr>
<tr>
<td>3-D</td>
<td>II</td>
<td>X-X</td>
<td>8.57</td>
<td>0.116</td>
</tr>
<tr>
<td>2-D</td>
<td>I</td>
<td>X-X</td>
<td>8.57</td>
<td>0.116</td>
</tr>
<tr>
<td>2-D</td>
<td>II</td>
<td>X-X</td>
<td>29.03</td>
<td>0.034</td>
</tr>
</tbody>
</table>

The first two modes obtained from the 2-D model are shown in Figure 8.4. The 2-D model showed the same frequency of the fundamental mode as the first mode in the X-X direction of the 3-D model. The second mode had a very high frequency and its influence would be insignificant on the model response. The frequencies and periods for the corresponding modes for both models are given in Table 8.1.

![Figure 8.4. First two mode shapes of the non-linear 2-D model (DRAIN-2DX).](image-url)
8.2.2 Time History Dynamic Analysis

Time history dynamic analyses were performed using both, the linear and non-linear analytical models. To enable direct comparison with the shake table test results, the actual shake table accelerations recorded in each test were used as the input ground acceleration for the analyses. The linear analysis did not include the nonlinear characteristics in the braces, so no comparison could be made with the experimental results on the brace level. The linear model, however, well represented the 3-D nature of the frame and the results were used for preliminary analysis. For example, a linear analysis with this model was first used before the shake table tests were conducted to obtain an estimate of the expected force levels in the braces.

As expected, the linear analysis showed higher acceleration levels at the top of the model in all simulations, with peak accelerations being up to 3 times higher than those recorded in the tests. In addition, the linear analytical model showed a higher frequency response with lower deformations at the top and higher forces generated in the brace. This observation supports the present seismic design philosophy, which recognizes the non-linear deformations and ductility as structure’s ability to dissipate energy and reduce the intensity of the seismic forces involved.

Non-linear analyses simulating most of the tests at different intensity levels using the DRAIN-2DX model were performed following the linear analyses. To permit comparison with the relative acceleration records generated by the DRAIN-2DX model, the shake table accelerations were subtracted from the accelerations measured at the top of the frame. For instance, a comparison of measured accelerations and deformations at the top of the north frame for test number 11 and the corresponding histories obtained analytically are presented in Figure 8.5.
Figure 8.5. A comparison between the measured and analytically obtained deformations and accelerations at the top of the north frame for test number 11.
As shown in Figure 8.5, the deformation history at the top of the frame was relatively well represented with the analytical model. Although the related peaks were about 30% lower than those obtained during the tests, their pattern closely resemble the one obtained during the test, with an exception of one big peak at approximately 10 seconds. The difference in the maximum deformations is mostly due to three main reasons. Firstly, the analytical model assumes all connections on the frame model to be pinned. Secondly, the slightly torsional response experienced during the shake table tests had an influence on the deformations of both north and south frame. This influence could not be represented in the 2-D DRAIN model. Thirdly, the difference is partially due to the limited abilities of the analytical model to represent the hysteretic behaviour of the braces at lower deformation levels. In other words, the response in the analytical model always starts with the initial brace stiffness (virgin curve) while in the actual tests the slackness in the brace connections is always present due to wood crushing and damage from the previous tests.

The analytical simulation also showed slightly higher maximum acceleration levels than the experimental ones. For example, the difference between the analytical and the measured values of test number 11 was about 15%. Similar differences were obtained for the other tests as well. These differences were much smaller than those obtained from the linear model. The inability to model the influence of the torsion and the higher initial stiffness in the analytical model were the main reasons for the differences.
Figure 8.6. A comparison between the measured and analytically obtained hysteresis loops in the braces of the north frame during test number 11.

Relatively good agreement was obtained when comparing the load-deformation relationship of the braces obtained analytically and during the tests. Figure 8.6 shows the obtained hysteresis curves of both braces from the analysis compared to the experimentally obtained loops for the braces of the north frame from test number 11. The difference between the model behaviour and
the experimental loops was again mainly due to reasons mentioned before, including the asymmetry of the frame response obtained from the shake table tests.

The damage caused in the connections (wood crushing) during one experimental test affected the response of the frame in the subsequent tests. The result was a larger initial slack in brace connections and lower initial stiffness, while the DRAIN model always captured the behaviour of the connection (brace) from its virgin state. By including the additional energy dissipation from wood crushing at lower deformation levels, the model also overestimated the dissipated energy by about 10%.

Results from the analytical study also showed that for obtaining more satisfactory results, the adjustment of the load-deformation characteristics of the braces in the model have to be done at different deformation levels, representing the different input acceleration levels. When an adjustment of the parameters in the braced model was done at larger deformation range, the response of the model at lower acceleration levels usually overestimated the experimental response.

A comparison of the measured and analytically obtained deformations and accelerations at the top of the north frame from test number 12 are presented in Figure 8.7. The brace load-deformation characteristics from the same test and their corresponding analytical simulations are shown in Figure 8.8.
Figure 8.7. A comparison between measured and analytically obtained deformations and accelerations at the top of the north frame for test number 12.
Figure 8.8. A comparison between measured and analytically obtained hysteresis loops in the braces of the north frame during test number 12.

8.3 SUMMARY

Mathematical models to simulate the behaviour of the braced timber frame specimen with riveted connections tested on a shake table are presented in this chapter. Results were presented from simulations with a 2-D non-linear model, developed using the DRAIN-2DX computer program. Although the program can perform non-linear dynamic analyses in two dimensions
only, it contains a subroutine that can simulate the specific type of non-linearity found in riveted connections. Typical load-deformation hysteretic curves for the braces obtained from shake table tests were used to model the non-linear behaviour in the frame. The results showed that the analytical model can reasonably predict the behaviour of the braced frame subjected to different earthquakes. The accelerations obtained at the top of the model were slightly higher than the values obtained during shake table tests, while the load-deformation response obtained in the braces obtained analytically was found to be close to the experimentally obtained behaviour.
9. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

9.1 SUMMARY

This dissertation has addressed the behaviour of concentrically braced timber frames as one type of system used to resist lateral loads caused by earthquakes. The background of the topic was discussed and the need for investigation of the dynamic behaviour of braced timber frames was demonstrated. The topic was then addressed from several perspectives, using experimental and analytical studies to determine the seismic design factors for braced frames with different connections. It can be stated that the objectives of the study were achieved and the study has yielded ample results, observations, comments and design guidelines that will be useful for design engineers in practice and for code officials.

The main objective in the first part of the experimental program was focused on characterising the seismic behaviour and failure modes of different connections used in braced timber frames, when subjected to a standard monotonic tension or cyclic loading. This was a necessary first step towards developing an analytical model for the prediction of the connection behaviour, and subsequently predicting the behaviour of a braced frame system under dynamic loads. Displacement controlled monotonic tension and cyclic tests were conducted on a number of connections with four different connector types, namely mild steel bolts of 9.5, 12.7 and 19 mm
diameter, and 65 mm long glulam rivets. Load-deformation static and hysteretic curves for all different connections were obtained and later used as input in the analytical modelling of braced frames.

In the second part of the experimental program, shake table tests were performed on full-scale single braced frame models. The components of the experimental test setup were described and testing procedures were discussed. A total of eight frames were tested utilizing five different connections, four of them tested quasi-statically. Test data were used to determine the influence of dynamic rate of loading on the load deformation characteristics of brace connections. In addition, test data were used for verification and calibration of the analytical models that were developed on the basis of quasi-static tests.

The third part of the experimental program consisted of a series of shake table tests on a two-storey braced frame model with riveted connections. The response of the braced frame model to three different earthquakes at different acceleration levels was observed and assessed. The natural frequencies of the model were obtained from impact hammer tests at various stages of testing. The large amount of data collected during the tests was used to describe the seismic response of braced timber frames with riveted connections and for verification of the analytical simulations of the frame response. Because failure of the model was not reached at the end of the shake table tests, pushover tests were conducted to determine the capacity of the structure. All specimens and models tested throughout the experimental program consisted of Spruce Pine Fir (SPF) glued laminated (glulam) members, 130 by 152 mm in cross section.
In the analytical part of the study, based on the results of the quasi-static and shake table tests, analytical models were developed for all different connections tested. These connection models were then incorporated in non-linear analytical models of braced frame structures. The structural models were then used in a series of non-linear static and dynamic analyses, using a selected group of different earthquakes as input motion. The braced frame models were calibrated based on the results from the shake table tests, and were subsequently used in a parametric study to evaluate the force modification factors (R-factors) for braced timber frames with different connections according to the National Building Code of Canada (NBCC).

9.2 CONCLUSIONS AND DESIGN RECOMMENDATIONS

The research presented in this dissertation is the first comprehensive attempt to produce seismic design factors from basic connection properties. The shake table tests were the first tests of its kind performed on a braced timber frame structure. The work presented can thus in many ways be considered as an important research contribution in the field of seismic behaviour of timber structures. The study has yielded ample results and conclusions on parameters that influence the seismic behaviour of braced timber frames. These findings will be useful for design engineers and code officials in improving the understanding of the fundamentals of the seismic response of braced timber frames. They can also be used by other researchers for refinement of procedures and expansion of the knowledge base. Some of the most important findings of the study are summarized as follows:

- Regardless of general misgivings about the use of concentrically braced frames in earthquake prone zones, it has been shown that braced timber frames can be used as
efficient lateral load resistance systems for earthquake loads in buildings. The study also confirmed that energy absorption capacity and overall ductility of the system, the two most important parameters for adequate seismic behaviour of braced timber frames, are almost entirely governed by the brace connections. This was evident from experimental as well as from analytical studies. It is therefore of paramount importance that the brace connections be designed and constructed to behave in a ductile manner for seismic loading.

- According to the principle of capacity design, the remainder of the frame has to be designed in a manner to avoid any distress in components that could potentially fail in a brittle mode. The brace connections have to be the weak link in the structure. This will ensure that the non-linear deformations take place in those connections, thus reducing the seismic forces.

- The issue of over-strength in fasteners has to be addressed when designing braced timber frames. Since commonly available bolts, for example, typically are made of relatively high strength carbon steel and often fail in a brittle manner, the fasteners for ductile connections in braced timber frames should be selected based on the minimum and maximum strength as well as ductility and energy dissipation.

- For the four types of connections investigated it was evident that the performance under earthquake loading was vastly different, although all of them would have satisfied current code requirements. The main issues that affected the varied response are the ductility of the connections and the consistency of their strength or stiffness parameters.
• Glulam riveted connections showed superior seismic performance when compared to bolted connections for similar design load levels. During quasi-static and shake table tests, glulam riveted connections exhibited a capability of resisting many load reversals without significant strength deterioration. In addition, large displacements were possible before failure, which permits ample warning before any potential structural failure. Braced timber frames with glulam riveted connections were able to dissipate the highest amount of seismic input energy generated by the earthquake motion.

• In braced frames with bolted connections it was shown that the behaviour is dependent on the bolt slenderness (ratio of the width of the wood member and the bolt diameter). In bolted connections with lower slenderness ratio (19 mm bolts), high flexural rigidity of the bolts resulted in more rigid connections with high stresses in the wood, which precipitated abrupt wood splitting and sudden loss of bearing capacity. Connections with bolts of higher slenderness ratios (smaller diameter bolts) exhibited a more desirable behaviour in that more wood crushing could occur before fracture, although eventually wood splitting was consistently the failure mechanism for all bolted connections tested. In addition, the energy dissipation in bolted connections with small diameter bolts was higher than in connections with larger diameter bolts. Based on these results it can be concluded that slender bolts are more desirable for seismic design of braced timber frames.

• Comparisons between the design loads and ultimate loads obtained for all connections from the quasi-static tests showed that the design load was between 36 and 59 % of the ultimate load. Bolted connections with two rows of bolts had a higher over-strength factor than the connections with one row of bolts, suggesting that the existing row
factor (0.8 for two rows of bolts) in the CSA O86.1-M94 design equations might be overly conservative. Riveted connections showed a lower over-strength factor than bolted connections, but lower variability.

• The study has shown that during a seismic event both connections in a single brace of the braced frame structure do not experience the same deformation levels. Due to numerous factors, including the variability of the strength properties of the wood, one of the brace connections starts to experience higher initial deformations and wood crushing, which results in concentration of the deformation demand for that connection later in the response. This finding is considered very important for understanding the seismic behaviour of braced timber frames, pointing out that the deformation capacity of a brace is not equal to the twice the capacity of one connection. This fact was later used when developing the analytical models for braced timber frames throughout the study.

• It was found that the dynamic rate of loading has an influence on the observed connection property parameters. Although the shake table setup and the earthquake chosen can influence the dynamic tests results, the envelope curves from monotonic tension tests overestimated the load and deformation characteristics of the connections subjected to earthquake ground motion. For that reason it is suggested that static tests should not be the sole source for determining the seismic response characteristics of a timber connection. Cyclic tests gave more realistic and reliable information on the connection seismic properties.

• The study has shown that for the design and construction of braced timber frames with bolted connections, the use of passive reinforcement, such as treadered rods, placed
transversally to the bolts, should be encouraged. This fast and inexpensive technique can significantly improve the seismic performance of any bolted connection used in braced timber frames. It can also be used for the retrofit of existing braces or repair of braces that suffered damage during previous earthquakes.

- Pin-ended attachments are often preferable for braces to avoid perpendicular to grain stresses in wood due to introduction of a rotation (moment) in the connection. Experimental tests showed, however, that it is difficult to avoid this rotation in brace connections and thus establish a purely axial stress state in the braces, even when pins are used at both ends of the brace. This is particularly the case for bolted connections with oversized holes. Glulam rivets proved to perform better in maintaining the alignment of the brace with respect to the pin-ended attachments.

- The nine-parameter “Florence” hysteresis model implemented in the DRAIN-2DX computer package is capable of predicting the seismic response of braced timber frames with good approximation, provided that the appropriate connection hysteresis parameters are used. The analytically obtained hysteresis response in the braces was found to closely resemble the experimental behaviour.

- Contrary to common belief that braced timber frames tend to be stiff structures, it has been shown that their initial natural periods are relatively long (from 0.4 to 0.75 sec), depending on the connections and the participating mass used for design. The initial periods obtained analytically were also found to be longer than the fundamental periods determined according to the NBCC recommendations (0.34 sec). Since the structural periods of vibration are prolonged during the actual seismic event, due to loosening of
the connections after the first few large cycles of motion, caution should be used when designing braced timber frames on soft soils in seismic regions.

- Damping values for braced timber frames were found to be between 2 % and 4 % of critical damping, depending on the deformation level and damage induced on the frame. Those values, along with the damping effects of the non-structural elements which were not accounted for in the study, are believed to be around 5 %. This is the value used in most seismic design codes.

- Braced timber frames with different connections should be assigned different force modification factors.

- Braced timber frames with bolted connections with slenderness ratios (l/d) of 10 or higher showed far more adequate seismic performance than frames that utilised bolts with lower slenderness ratios. Until further research is undertaken (to study the effects of connection parameters on the seismic behaviour) and general recommendations are developed that will ensure ductile behaviour of braced frames with bolted connections, an R-factor of 1.5 is suggested for braced timber frames with slender bolts. In no case, however, should the seismic weight exceed four times the brace connection design force.

- Riveted connections were the only connections tested that consistently showed non-brittle deformations in the wood along with yielding of the connectors, even at large displacement levels. The study showed that glulam riveted connections designed in rivet yielding mode can be assigned an R factor of 2.0, in recognition of their higher
and more consistent ductility capacity. For frames with riveted connections the seismic weight can be up to 4.5 times the brace connection design force.

- Narrow braced frames, with aspect ratios (storey height vs. frame width) higher than one, should be avoided because of their cantilever bending type response. Wider frames have been shown to exhibit more of a shear type response and make better use of the braces and their connections. Very wide frames with aspect ratios lower than 0.67, however, should be avoided because the benefits of having a wider frame are usually outweighed by the drawbacks of having a long brace, susceptible to buckling at lower force levels.

- The seismic demands for multi-storey braced frames were shown to be either at the same level or slightly higher than the corresponding demands for the single storey frames. The increased demands, however, were not significant enough to warrant different recommended R-factor values.

9.3 RECOMMENDATIONS FOR FUTURE RESEARCH

During this study a large amount of valuable information was gathered on the seismic behaviour of braced timber frames. The results, however, have also shown the need for further research to investigate a number of different topics pertaining to braced timber frames. Some of the topics that should be further investigated are the following:

- In this study, only braced timber frames with glued-laminated timber were addressed, with one size of cross section. It is important to study other materials that can be
potentially used in braced frames such as Parallel Strand Lumber (PSL) and Laminated Veneer Lumber (LVL).

- Further research should be undertaken to study the effects of parameters such as end distance, spacing, number of rows, number of bolts in a row etc. Based on these findings, general recommendations should be developed to ensure ductile behaviour of braced frames with bolted connections.

- The effect of type and rate of loading on a connection hysteresis curves should be established for different connectors and material properties.

- Further research is needed to determine ways of improving the seismic behaviour of bolted connections in braced frames. One of the options is to study the improvement that reinforced connections can bring to the seismic response of braced timber frames. In addition, different types of brace connections should be investigated including connections with tight fitting dowels and hollow tubes.

- In the analytical part of the study five different earthquake records were used with zonal characteristics of a seismic site such as Vancouver. More earthquake motions could be used to cover a wider range of possible input motions for the same site, as well as using records that are characteristic for different sites, especially in eastern Canada.

- More sophisticated computer software can be used to predict the 3-D seismic response of a timber structure with braced frames as lateral load resistant system. The option of bi-directional input to the structure should be studied.
• The effects of different rigidities of the horizontal diaphragms on the seismic response of braced timber frames should be addressed. In this study it was assumed that the roof diaphragms were stiff enough to transfer the horizontal load to the braced frames according to their stiffness.

• A reliability analysis should be performed to determine the R-factors for braced timber frames. This method will provide a structural safety evaluation in terms of probability. The basic random variables, including the PGA for the input acceleration should be identified and their basic statistics should be obtained. A non-linear analytical model, developed in a computer program for performing time history dynamic analysis such as DRAIN can be used to obtain the seismic demand. That program should be linked with a reliability program to determine the probability of failure for a particular braced frame subjected to a number of different earthquake records. The event of failure can be defined as exceeding a specific amount of deformation demand in the brace connections. This concept can determine the value of an R-factor for a given target safety level.
10. BIBLIOGRAPHY


APPENDIX A

A VERSION OF THE SPREADSHEET USED FOR SIGNAL PROCESSING

Before the data recorded from the sensors placed on the model can be used for quantification of the dynamic response, a detailed analysis of the signals in the time and frequency domain (pre-processing) has to be performed. The pre-processing was performed using a spreadsheet developed for that purpose in the commercial mathematical software package Mathcad. A printout of one version of the spreadsheet is given here.

Preparation of Braced Frame Model Data

Define array sizes:

No. of channels: \( N_c := 10 \)  
Time Increment: \( \Delta := 0.005 \)

Duration of the record: \( T_d := 59.9 \Rightarrow N := \frac{T_d}{\Delta} \Rightarrow N = 1.198 \cdot 10^4 \)

\( i := 0 \ldots N - 1 \)  
\( j := 0 \ldots N_c - 1 \)  
\( t_i := i \cdot \Delta \)  
\( k := 0 \ldots N_c + 1 \)

UNF\(_{i,j}\) := READ("run15.asi")  
d := UNF\(^{<1>}\)

\[\begin{array}{c}
d_i \\
0 \\
-5 \\
0 \\
2000 \\
4000 \\
6000 \\
8000 \\
1 \cdot 10^4 \\
1.2 \cdot 10^4
\end{array}\]
$F_d := \text{mag}(\text{cfft}(d))$

$$m := 0 \ldots \frac{N}{2}$$

$$f_m := \frac{1}{N \Delta}$$

Number of points used for averages

$$\text{Navr} = 50$$

$$\sum_{i=0}^{\text{Navr}-1} \text{UF}_{i,j}$$

$$\text{UFavr}_{j} := \frac{\sum_{i=0}^{\text{Navr}-1} \text{UF}_{i,j}}{\text{Navr}}$$

$$\text{UF}_{i,j} := \text{UNF}_{i,j} - \text{UFavr}_{j}$$

Window the signal:

$$\text{Win} := \text{taprect}(N)$$

$$\text{UF}_{i,j} := \text{UF}_{i,j} \cdot \text{Win}_i$$

$$e := \text{UF}^4$$

$e_i$

$\text{d}_i$
Filtering the input record to remove any of the high and low frequencies:

First setup our filter

\[
\begin{align*}
\text{Cutoff} & := 0.075 & \text{Cutfreq} & := \frac{\text{Cutoff}}{\Lambda} & \text{Cutfreq} & = 15 \\
\text{Cutoff1} & := 0.00125 & \text{Cutfreq1} & := \frac{\text{Cutoff1}}{\Lambda} & \text{Cutfreq1} & = 0.25
\end{align*}
\]

\[
\begin{align*}
L & := \text{iirlow(butter(4), Cutoff)} & \text{xf} & := 0, 0.001 \ldots .5 & M & := \text{iirhigh(butter(2), Cutoff1)}
\end{align*}
\]

\[
\begin{align*}
\text{FHL(Matrix)} & := \text{for } j \in 1 \ldots Nc - 1 \\
& \quad \text{col} \leftarrow \text{Matrix}^<j> \\
& \quad \text{highpass} \leftarrow \text{response} (\text{col}, M, N) \\
& \quad A \leftarrow \text{response} (\text{highpass}, L, N) \\
& \quad F^<j> \leftarrow A
\end{align*}
\]

\[
\begin{align*}
\text{FUil} & := \text{FHL(UF)} \\
C & := 7 \\
\text{v1} & := \text{UNF}^<C> \\
\text{v2} & := \text{FUil}^<C> \\
\text{i} & := 0 \ldots N - 2 \\
\text{fv1} & := \text{mag}(\text{cfft(v1)}) \\
\text{fv2} & := \text{mag}(\text{cfft(v2)}) \\
\text{k} & := 0 \ldots \frac{N - 2}{2} \\
\text{f_k} & := \frac{k}{(N - 2) \cdot \Lambda}
\end{align*}
\]

\[
\begin{align*}
\text{max(UNF}^<C>) & = 1.528 \cdot 10^4 & \text{min(UNF}^<C>) & = -1.584 \cdot 10^4 \\
\text{max(FUil}^<C>) & = 1.615 \cdot 10^4 & \text{min(FUil}^<C>) & = -1.727 \cdot 10^4
\end{align*}
\]
Print files as individual channels:

Channels can be printed as UF (unfiltered), FH (Filtered high pass), FHL (Filtered low and high pass)

```
WRITEPRN("channel0.prn") := UFIi<0>
WRITEPRN("channel1.prn") := UFIi<1>
WRITEPRN("channel2.prn") := UFIi<2>
WRITEPRN("channel3.prn") := UFIi<3>
WRITEPRN("channel4.prn") := UFIi<4>
WRITEPRN("channel5.prn") := UFIi<5>
WRITEPRN("channel6.prn") := UFIi<6>
WRITEPRN("channel7.prn") := UFIi<7>
WRITEPRN("channel8.prn") := UFIi<8>
WRITEPRN("channel9.prn") := UFIi<9>
WRITEPRN("channel10.prn") := UF<10>
WRITEPRN("ffurier.prn") := fv2
WRITEPRN("unfurier.prn") := fv1
```