Behaviour and Design of Generic Buckling Restrained Brace Systems

by

Stefan Wijanto

A thesis submitted in fulfilment of the requirement for the degree of

Master of Engineering

Supervised by

Associate Professor Charles G. Clifton

Department of Civil and Environmental Engineering

The University of Auckland

Auckland

New Zealand

February 2012
Abstract

The recent series of damaging earthquakes in Christchurch, New Zealand (NZ) has encouraged greater recognition of the post-earthquake economic impacts on NZ society and higher emphasis on low-damage earthquake resisting systems. Buckling Restrained Braces (BRB) are seen as a significant contender for such a system. They have been developed and used in both North America and Japan and are recognised for their superior seismic performance compared to existing concentrically braced systems due to the suppression of brace buckling in compression, and hence the development of equal strength and stiffness under tension and compression loading. However, the focus of development in those countries has been on establishing a testing regime to which companies produce patented systems. This has limited their application in New Zealand due to small demand and has generated interest in development of a generic solution.

This research project focuses on the development of a reliable design procedure and detailing requirements for a generic BRB system. This started with the development of a design procedure based on modifications of the concentrically braced frame (CBF) design procedure contained in HERA Report R4-76 (1995). This has been used to develop a representative design for a 10 storey building, from which a brace size and bay has been chosen for experimental testing. A series of dynamic sub-assemblage tests were performed at the University of Auckland on this BRB frame with two different brace connection configurations to gauge the performance of the designed system. The results are presented and discussed herein. An initial prototype model for analytical modelling of the sub-assemblage frame has also been constructed and subjected to inelastic time history analyses.

The experimental tests show stable hysteresis loops which is a principal feature of the BRB system, albeit with the occurrence of slack in the connections. These test results show the reliability of the proposed design procedure and detailing, especially after procedural modifications to prevent slack from occurring in the two different connection systems.
Acknowledgements

This thesis would not have been possible without the tremendous assistance provided by the individuals and organisations mentioned below. I would like to gratefully acknowledge them for their incredible amount of dedication and support, which has enabled the successful completion of this research and report. In particular I would like to acknowledge the following individuals:

- My supervisors, Associate Professor Charles Clifton for his comprehensive guidance and support, and my second supervisor Associate Professor John Butterworth for his advice throughout the research period;
- The Auckland University technicians, especially Jeffery Ang whose expertise and dedication enabled me to complete my experimental project;
- My family, for their endless support and encouragement throughout my academic career;
- My colleagues, especially Hsen-Han Koo and John O’Hagan for their help on the experimental phase of my project;
- My friends, who made this journey so much more enjoyable, and especially Xiaoming Wang and Yida Zou for formatting and proofreading my thesis.

I would like to also acknowledge the contribution made by the following persons and organisations towards this project:

- Alistair Fussell of SCNZ, for providing critical information for this research project;
- Professor Emeritus Athol Carr of the University of Canterbury, for technical advice and guidance on using the analysis program RUAUMOKO and compiling the Christchurch Earthquake digital record used for analytical modelling;
- Earthquake Commission (EQC) New Zealand, for their financial assistance;
- The companies who contributed the supply of materials and fabrication of specimens at reduced or no cost;
  - Grayson Engineering (Mike Moore and Rod Wallace)
  - Sika NZ (Mike Edwards)
  - Denso Pty Ltd (Wayne Thompson)
Table of Contents

Abstract ........................................................................................................................................ i
Acknowledgements ................................................................................................................... iii
List of Figures ............................................................................................................................ xi
List of Tables .......................................................................................................................... xvii
List of Symbols ........................................................................................................................ xix
Chapter 1 INTRODUCTION ..................................................................................................... 1
  1.1 Background ....................................................................................................................... 1
  1.2 Objectives ......................................................................................................................... 3
  1.3 Research Methodology and Timeline ............................................................................... 3
  1.4 Outline of Thesis .............................................................................................................. 5
Chapter 2 LITERATURE REVIEW .......................................................................................... 7
  2.1 Introduction to Literature Review .................................................................................... 7
  2.2 Concept of BRB ............................................................................................................... 8
    2.2.1 Conception ................................................................................................................. 8
    2.2.2 Anatomy of a Buckling Restrained Braces System ............................................... 10
  2.3 Development of BRB ..................................................................................................... 11
    2.3.1 Timeline ................................................................................................................... 11
    2.3.2 Japan ........................................................................................................................ 11
    2.3.3 United States ............................................................................................................ 12
    2.3.4 New Zealand ............................................................................................................ 13
  2.4 Design and Testing Requirement ................................................................................... 13
    2.4.1 United States Requirement ...................................................................................... 13
    2.4.2 New Zealand Requirement ...................................................................................... 14
  2.5 Other Analytical and Experimental Testing Studies ...................................................... 14
4.3.2 Gap Bearing ............................................................................................................. 72
4.3.3 Contraction Allowance ............................................................................................ 73
4.4 Construction Material Specification ............................................................................... 73
  4.4.1 Steel ......................................................................................................................... 73
  4.4.2 De-bonding agent .................................................................................................... 75
  4.4.3 Filler Material .......................................................................................................... 76
4.5 Construction Phase ......................................................................................................... 77
  4.5.1 Trial Erection ........................................................................................................... 77
  4.5.2 Gap Bearing ............................................................................................................. 77
  4.5.3 Denso Tape Wrapping ............................................................................................. 78
  4.5.4 Grout Pouring .......................................................................................................... 80
Chapter 5 SUB-ASSEMBLAGE TESTING ............................................................................ 83
  5.1 Introduction .................................................................................................................... 83
  5.2 Installation and Test Setup ............................................................................................. 84
    5.2.1 Overall Test Set Up ................................................................................................. 84
    5.2.2 Restraining Mechanism ........................................................................................ 85
    5.2.3 Instrumentation ...................................................................................................... 86
    5.2.4 Pinned Specimen .................................................................................................... 87
    5.2.5 Bolted Specimen ................................................................................................... 88
  5.3 Bare Frame Test .............................................................................................................. 89
    5.3.1 Introduction ............................................................................................................. 89
    5.3.2 Loading Regime ...................................................................................................... 89
    5.3.3 Observations ............................................................................................................ 90
    5.3.4 Results and Comparison .......................................................................................... 90
    5.3.5 Discussion .............................................................................................................. 91
  5.4 Trial Static Test .............................................................................................................. 92
    5.4.1 Introduction ............................................................................................................. 92
6.2.5 Weights and Loads ................................................................. 131
6.3 Analysis Results ........................................................................ 131
   6.3.1 Analysis Options ................................................................. 131
   6.3.2 Time History Records......................................................... 132
   6.3.3 Hysteresis Results............................................................... 132
Chapter 7 CONCLUSIONS ............................................................. 135
   7.1 Principal Objective Conclusions ............................................ 135
      7.1.1 Design Procedure.......................................................... 135
      7.1.2 Design and Detailing of BRB Elements ......................... 136
      7.1.3 Experimental Tests ........................................................ 136
      7.1.4 Time History Analyses .................................................. 137
   7.2 Recommendations for Future Testing and Research .............. 138
Chapter 8  References .................................................................... 139
APPENDIX A DESIGN PROCEDURE EXAMPLES ......................... 143
   A.1 Spectral Shape Factor Calculation ...................................... 143
   A.2 Base Shear Calculation ....................................................... 144
   A.3 Equivalent Static Force Calculation ................................. 145
   A.4 Wind Force Calculation ..................................................... 147
   A.5 Gravity Column Sizing ....................................................... 149
   A.6 Period Determination and Lateral Deflection Check ........... 151
   A.6 Simply Supported BRB Design ......................................... 153
   A.7 Propped Cantilever BRB Design ....................................... 153
   A.8 Collector Beam Design ..................................................... 154
   A.9 Seismic Column Design .................................................... 156
APPENDIX B Detailing Procedure Examples ................................. 157
   B.1 Gusset Plate Design Example .......................................... 157
   B.2 Moment Endplate Design Example .................................... 160
B.3 Bolt and Pin Design Example ................................................................. 161

APPENDIX C CONSTRUCTION DRAWINGS .............................................. 163
  C.1 List of Quantities .................................................................................. 163
  C.2 Assembly Drawing .................................................................................. 165
  C.3 Parts Drawings ....................................................................................... 175

APPENDIX D MATERIAL SPECIFICATIONS .............................................. 189
  D.1 Steel Tensile Test .................................................................................. 189
  D.2 Grout Specification ................................................................................. 190
  D.3 Denso Tape Specification ....................................................................... 194

APPENDIX E RUAUMOKO Input ................................................................. 195
  E.1 Sub Assemblage Bolted ......................................................................... 195
  E.2 Sub Assemblage Pinned ......................................................................... 199
List of Figures

Figure 1.1: Fractured EBF (left) and Buckled CBF (right) (Bruneau et al., 2011) .................... 1
Figure 1.2: Examples of BRBs system at UC Berkeley ............................................................. 2
Figure 2.1: Behaviour of conventional brace and BRB (Xie, 2005) ......................................... 8
Figure 2.2: Mechanics of BRB (Lopez & Sabelli, 2004) ........................................................... 9
Figure 2.3: Anatomy of BRB element minus connection and projected section (Sabelli et al., 2003) ......................................................................................................................................... 10
Figure 2.4: Timeline of BRB system development (Ko and Claire) ....................................... 11
Figure 2.5: BRB test setup and its hysteresis (Wakabayashi et al., 1973) ............................... 11
Figure 2.6: BRB test setup and its hysteresis response (Hussain et al., 2005) ......................... 12
Figure 2.7: (a) Large scale test setup and (b) beam-column-brace connection details (Fahnestock et al., 2007) ..................................................................................................................................... 15
Figure 2.8: Experimental testing and analytical modelling results (Tremblay et al., 2004) .... 16
Figure 2.9: Experimental testing and analytical modelling results (Deulkar et al., 2010) ...... 17
Figure 2.10: Comparison of displacement between different types of BRB bracing configurations (Deulkar et al., 2010) .............................................................................................................. 17
Figure 2.11: Comparison of hysteresis result between different types of filler material (Gheidi et al., 2010) ............................................................................................................................... 18
Figure 2.12: Comparison of hysteresis results between BRBs with different encasing tube thickness (Young et al., 2009) ........................................................................................................ 19
Figure 2.13: Different BRB cross-sections (Xie, 2005) ......................................................... 20
Figure 2.14: Standard bolted connection; (b) modified bolted connection, and (c) true pinned connection (Xie, 2005) ................................................................................................................. 22
Figure 2.15: Proposed configurations of BRB connections (Xie, 2005) ....................... 22
Figure 3.1: Typical floor plan ................................................................................................... 27
Figure 3.2: Elevation of building ............................................................................................ 27
Figure 3.3: Model Elevation .................................................................................................. 32
Figure 3.4: Difference between yield length and work-to-work point length (Fussell, 2010b) .............................................................................................................................................. 34
Figure 3.5: Mode 1 (left), T = 2.45 s and mode 2 (right), T = 0.83 s of the study case building analysis result ................................................................................................................................. 36
Figure 3.6: Final design lateral force (left) and deflected shape (right) of the study case building analysis result................................................................. 36
Figure 3.7: Comparison of fundamental period obtained by standard empirical formulae and analysis results (SCNZ, 2011) ................................................................. 37
Figure 3.8: Comparison between a simply supported and propped cantilever design........ 39
Figure 3.9: Different sections of a BRB element........................................................................ 40
Figure 3.10: Collector beam axial design calculation for inverted V-braced configuration with modification (HERA report R4-76, 1995) ........................................................................ 43
Figure 3.11: Distance of L’ and L and typical beam-brace connection for inverted V-braced configuration (HERA report R4-76, 1995) ........................................................................ 45
Figure 4.1: BRB elements (Sabelli and Lopez, 2004) .................................................................... 56
Figure 4.2: Main plate of the bolted specimen.......................................................................... 58
Figure 4.3: Summary of cross-section output ........................................................................... 61
Figure 4.4: (a) Load distribution of a pin in bearing and (b) configuration of a generic pinned specimen................................................................................................. 62
Figure 4.5: Clamping plate dimension ......................................................................................... 63
Figure 4.6: Pinned connection details ........................................................................................... 64
Figure 4.7: Additional details of the pinned connection ................................................................ 64
Figure 4.8: Deflected shape ......................................................................................................... 65
Figure 4.9: Bolted connection details ............................................................................................ 66
Figure 4.10: Welding between gusset plates ................................................................................ 66
Figure 4.11: (a) Pinned connection gusset plate and (b) bolted connection gusset plate ........ 68
Figure 4.12: Moment endplate details .......................................................................................... 70
Figure 4.13: Welded column details ............................................................................................ 71
Figure 4.14: Brace End with Gap Bearing .................................................................................... 72
Figure 4.15: Deflected shape of the subassembly model .............................................................. 72
Figure 4.16: Tensile test specimen dimensions .............................................................................. 74
Figure 4.17: (a) Tensile test machine and (b) broken specimens samples compared to the initial condition .............................................................................................. 74
Figure 4.18: Tensile test results of displacement vs. axial load ................................................. 74
Figure 4.19: Partially erected, bolted and pinned specimen ......................................................... 77
Figure 4.20: Polystyrene placement (left) and wrapping (right) .................................................... 78
Figure 4.21: Overlap sheets of Denso tape (left) and final look after (right) ............................. 79
Figure 4.22: Before (left) and after the Denso tape wrapping (right) ........................................... 79
Figure 4.23: Pinned specimen cover (left) and bolted specimen cover (right) ......................... 80
Figure 4.24: Sika Grout 212 Package (left) and grout mix (right) ........................................... 81
Figure 4.25: Grout pouring rig .................................................................................................. 81
Figure 4.26: Pouring grout using bucket (left) and fully grouted specimens (right) ............... 81
Figure 4.27: Results after 3 days .............................................................................................. 82
Figure 5.1: Elevation plan of the test set up ............................................................................. 84
Figure 5.2: Floor plan of the test set up .................................................................................... 85
Figure 5.3: Stress bar on left column (left) and initial lateral restraint (right) ......................... 85
Figure 5.4: Actuator (left) and LVDT on the top back column (right) .................................... 86
Figure 5.5: Portal gauge connected to the right column (left) and turn pot to measure in-plane movement (right) ...................................................................................................................... 86
Figure 5.6: Actuator controller (left) and data logger processed data (right) ........................... 87
Figure 5.7: Installed pinned specimen ...................................................................................... 87
Figure 5.8: Installed bolted specimen ....................................................................................... 88
Figure 5.9: Bare frame test ....................................................................................................... 89
Figure 5.10: Hysteresis result of bare frame test ...................................................................... 90
Figure 5.11: Pull force reduction graph .................................................................................... 91
Figure 5.12: Push force reduction graph ................................................................................ 92
Figure 5.13: Mode of failures .................................................................................................. 93
Figure 5.14: Extra reinforcements ........................................................................................... 94
Figure 5.15: (a) Actuator pushing the frame sideways and (b) new restraining mechanism ... 94
Figure 5.16: Stand-alone LVDT measuring the frame movement ........................................ 95
Figure 5.17: Hysteresis of pinned specimen for trial static test .............................................. 97
Figure 5.18: Hysteresis result of bolted specimen for trial static test .................................... 98
Figure 5.19: Lateral deformation vs. axial load of pinned specimen before reinforcement .... 99
Figure 5.20: Lateral deformation vs. axial load of pinned specimen after reinforcement ....... 100
Figure 5.21: Cyclic static test loading regime for pinned specimen ...................................... 101
Figure 5.22: Cyclic static test loading regime for bolted specimen ........................................ 101
Figure 5.23: Cyclic static tests set up for pinned specimen (left) and bolted specimen (right) ................................................................................................................................................ 102
Figure 5.24: Raw data of pinned specimen for cyclic static test ............................................ 104
Figure 5.25: Hysteresis of bolted specimen for cyclic static test ........................................... 104
Figure 5.26: Raw data of bolted specimen for cyclic static test ............................................ 105
Figure 5.27: Hysteresis of bolted specimen for cyclic static test ........................................... 105
Figure 5.28: Comparison of bolted and pinned cyclic static test results (D_{by}) ................. 106
Figure 5.29: Comparison of bolted and pinned cyclic static test results (D_{bm})............... 106
Figure 5.30: Comparison of bolted and pinned cyclic static test result (1.25 D_{bm}) ........... 107
Figure 5.31: Movement of pinned cyclic specimen centre .................................................... 107
Figure 5.32: Movement of bolted cyclic specimen centre ................................................... 108
Figure 5.33: Cyclic dynamic loading of pinned specimen .................................................... 111
Figure 5.34: Cyclic dynamic loading of bolted specimen .................................................... 112
Figure 5.35: Dynamic test set up ........................................................................................... 113
Figure 5.36: Raw data of pinned specimen for cyclic dynamic tests ..................................... 114
Figure 5.37: Hysteresis of pinned specimen for cyclic dynamic tests ................................... 115
Figure 5.38: Raw data of bolted specimen for cyclic dynamic tests ..................................... 115
Figure 5.39: Hysteresis of bolted specimen for cyclic dynamic tests ................................... 116
Figure 5.40: Comparison of bolted and pinned cyclic dynamic test result (D_{by}) .............. 116
Figure 5.41: Comparison of bolted and pinned cyclic dynamic test results (D_{bm}) ............ 117
Figure 5.42: Comparison of bolted and pinned cyclic dynamic test results (1.25 D_{bm}) .... 117
Figure 5.43: Comparison of subsequent loading of bolted specimen for cyclic dynamic tests (D_{bm}) ........................................................................................................ 118
Figure 5.44: Comparison of bolted specimen results on static and dynamic tests (1.25 D_{bm}) 119
Figure 5.45: Strain ageing test results of pinned specimen ................................................... 119
Figure 5.46: Steel behaviour .................................................................................................. 120
Figure 5.47: Post mortem of both specimens ........................................................................ 122
Figure 5.48: Setup and BRB hysteresis results obtained by Dinu et al. (2004) ................. 124
Figure 5.49: (a) Setup and (b) retrofitted frame hysteresis results obtained by Dinu et al. (2004) ........................................................................................................................................ 125
Figure 6.1: Overview of the initial BRB model ................................................................. 128
Figure 6.2: Steel beam-column yield interactions (Carr, 2008a) ....................................... 130
Figure 6.3: (a) Bi-linear Hysteresis and (b) Ramberg-Osgood Hysteresis (Carr, 2008b) .... 130
Figure 6.4: Hysteresis of steel core obtained from RUAUMOKO sub-assemblage analysis 132
Figure 6.5: BRB behaviour modelled using Ramberg-Osgood Hysteresis (Tremblay et al., 2004) ...................................................................................................................................... 133
Figure C.6: Full Assembly ................................................................................................. 166
Figure C.7: Laboratory Floor Plan ..................................................................................... 167
Figure C.8: Detail A - Bolted Specimen ............................................................................. 167
Figure C.9: Detail B - Bolted Specimen

Figure C.10: Detail A - Pinned Specimen

Figure C.11: Detail B - Pinned Specimen

Figure C.12: Additional Pinned Specimen Details

Figure C.13: Additional Endplate Weld Detail

Figure C.14: Detail C

Figure C.15: Detail D

Figure C.16: Actuator Connection to Strong Wall

Figure C.17: Actuator Connection to Frame

Figure C.18: Item 1 – Beam 360UB50.7

Figure C.19: Item 2a – Left Column

Figure C.20: Item 2b – Right Column

Figure C.21: Item 3 – Moment Flush Endplate

Figure C.22: Item 4 – Stiffener

Figure C.23: Item 5a – Left Baseplate to Column

Figure C.24: Item 5b – Left Baseplate to Floor

Figure C.25: Item 6 – Right Baseplate

Figure C.26: Item 7 – Actuator Connection to Strong Wall

Figure C.27: Item 8 – Actuator Connection to Frame

Figure C.28: Item 9/16 – Endplate Bottom

Figure C.29: Item 10/17 – Endplate Side

Figure C.30: Item 11 – First Plate Bolted

Figure C.31: Item 11 – First Plate Bolted

Figure C.32: Item 12a – Second Plate (Gusset)

Figure C.33: Item 12b – Second Plate BRB

Figure C.34: Item 13 – Connecting Plates

Figure C.35: Item 14 – Gusset Plate

Figure C.36: Item 14- Gusset Plate

Figure C.37: Item 14 - BRB Tube

Figure C.38: Item 9/16 - Endplate Bottom

Figure C.39: Item 10/17 – Endplate Side

Figure C.40: Item 18 – Pin

Figure C.41: Item 19 – First Plate
Figure C.42: Item 20 – Second Plate Pinned ................................................................. 187
Figure C.43: Item 21 – Gusset Plate Pinned ................................................................. 187
Figure C.44: Item 22 – Clamping Plate ...................................................................... 188
Figure C.45: Item 23 – BRB Tube ............................................................................. 188
Figure D.46: Tensile Test Results............................................................................. 189
List of Tables

Table 1.1: Research Timetable ................................................................................................... 4
Table 2.1: Different types of BRB Connection (Xie, 2005).......................................................... 21
Table 3.1: Relationship between structure category and member category for BRB Systems (NZS3404.7, 2011) .................................................................................................................. 28
Table 3.2: Load Applied during Preliminary Design ............................................................... 30
Table 3.3: Preliminary Design Force (kN) ............................................................................... 31
Table 3.4: Summary of different model options analysis (mm).................................................. 35
Table 3.5: Summary of final deflection results and drift ratio .............................................. 38
Table 3.6: Summary of axial design forces of beams in kN .................................................... 44
Table 3.7: Summary of beam seismic shear forces in kN ...................................................... 46
Table 3.8: Summary of column seismic axial forces in kN .................................................. 48
Table 3.9: Simply Supported – Beam and Column Sizes ......................................................... 49
Table 3.10: Simply Supported – BRB Size (Cruciform Shape in mm) ..................................... 49
Table 3.11: Propped Cantilever – Beam and Column Sizes ................................................... 50
Table 3.12: Propped Cantilever – BRB Size (Cruciform Shape in mm) .................................. 50
Table 5.1: Maximum forces subjected to the specimen in different loading regimes .......... 121
Table B.1: Preliminary Design of Gusset Plate ...................................................................... 157
Table B.2: Gusset Plate Design .............................................................................................. 158
Table B.3: Moment Endplate Design ..................................................................................... 160
Table B.4: Bolt design ........................................................................................................... 161
Table B.5: Pin Design ........................................................................................................... 161
Table C.1: Schedule List of Quantities .................................................................................. 163
Table D.1: Calculation of Yield Strength .............................................................................. 189
List of Symbols

A Variable used to account for the deterioration in inelastic performance of CBF with increasing brace slenderness

$A_d$ Post-deformation cross-section area of the brace

$A_i$ Initial cross-section area of the brace

$A_s$ Cross-section area

$A_{sc}$ Cross-section area of BRB steel core

B Variable used to account for the departure of the CBF system from the optimum weak beam strong column (overall) mechanism towards the less desirable strong beam – weak column (shear) mechanism

$C_h(T_1, \mu)$ Basic seismic hazard acceleration coefficient from NZS4203, being a function of the translational period of vibration, $T$, and the structural ductility factor, $\mu$

$C_s$ Factor applied to obtain design seismic load for CBF system

$D_b$ Deformation quantity used to control loading of Test Specimen (total brace end rotation for the Sub-assemblage Test Specimen; total brace axial deformation for the Brace Test Specimen)

$D_{bm}$ Value of deformation quantity, $D_b$, corresponding to the Design Storey Drift

$D_{by}$ Value of deformation quantity, $D_b$, at first significant yield of Test Specimen

E Young’s modulus, a measure of stiffness in solid mechanics

$E$ Earthquake force for the ultimate limit state

$f_y$ Yield strength of the steel component

$F_{beam}^c$ Net seismic vertical force applied to the collector beam in a V-braced CBF

$F_{beam,i}^c$ Net seismic vertical force applied to the collector beam in a V-braced CBF at level I of the structure

$h_i$ Height from the base of the structure to level i

$h_i - h_{i-1}$ The inter-storey height for storey i

$h_n$ Height from the base of the structure to the top of the structure, level n

$H_{beam,brace,com}^*$ Horizontal part of compression force from brace into column
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_C$</td>
<td>Column second moment of area</td>
</tr>
<tr>
<td>$I_{\text{min}}$</td>
<td>Minimum required second moment of area</td>
</tr>
<tr>
<td>$k_e$</td>
<td>Member effective length</td>
</tr>
<tr>
<td>$k_p$</td>
<td>Factor for pin rotation</td>
</tr>
<tr>
<td>$k_t$</td>
<td>Variable used to account for different brace system (see NZS1170.5 Commentary Clause C4.1.2.2)</td>
</tr>
<tr>
<td>$K_\theta$</td>
<td>Spring rotational stiffness</td>
</tr>
<tr>
<td>$L$</td>
<td>Member length for assessment of slenderness in compression</td>
</tr>
<tr>
<td>$L_{b,\text{max}}$</td>
<td>Maximum deformed brace length from work point to work point</td>
</tr>
<tr>
<td>$L_c$</td>
<td>Distance between adjacent column centres</td>
</tr>
<tr>
<td>$L_d$</td>
<td>After deformation area of the brace</td>
</tr>
<tr>
<td>$L_i$</td>
<td>Initial length area of the brace</td>
</tr>
<tr>
<td>$L'$</td>
<td>(Shorter) distance between the column face and critical sections of the collector beam in a V-braced CBF</td>
</tr>
<tr>
<td>$M_{\text{beam,GQ}_u}$</td>
<td>Bending moment in a beam due to long term gravity load combinations G &amp; Q$_u$</td>
</tr>
<tr>
<td>$M_{\text{casing}}$</td>
<td>Nominal moment capacity of the BRB casing</td>
</tr>
<tr>
<td>$M_{rx}$</td>
<td>Nominal moment capacity (about the major principal x-axis) of a section, reduced by axial forces</td>
</tr>
<tr>
<td>$M_{sx}$</td>
<td>Nominal section moment capacity about the x-axis</td>
</tr>
<tr>
<td>$M^*$</td>
<td>Design bending moment in a beam</td>
</tr>
<tr>
<td>$N_{\text{brace,E}}$</td>
<td>Brace seismic axial force due to load case E</td>
</tr>
<tr>
<td>$N_{\text{brace,GQ}_u}$</td>
<td>Brace axial force due to long term gravity load combinations G &amp; Q$_u$</td>
</tr>
<tr>
<td>$N_c$</td>
<td>Nominal member capacity in compression</td>
</tr>
<tr>
<td>$N_t,\text{brace}$</td>
<td>Nominal member capacity of a brace in tension</td>
</tr>
<tr>
<td>$N_{c,\text{brace}}$</td>
<td>Nominal member capacity of a brace in compression</td>
</tr>
<tr>
<td>$N_{\text{col,GQ}_u}$</td>
<td>Column axial (compression) force due to long term gravity load combinations G &amp; Q$_u$</td>
</tr>
<tr>
<td>$N_t$</td>
<td>Nominal section capacity in tension</td>
</tr>
<tr>
<td>$N_t,\text{brace}$</td>
<td>Nominal section capacity of a brace in tension</td>
</tr>
<tr>
<td>$N_{tf}$</td>
<td>Nominal tension capacity of bolt</td>
</tr>
<tr>
<td>$N_{\text{beam,i}}^c$</td>
<td>Capacity design-derived axial force in a beam at level i</td>
</tr>
<tr>
<td>$N_{c,l}^c$</td>
<td>Capacity design-derived axial load</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$N_{\text{col}}^c$</td>
<td>Capacity design-derived column axial force</td>
</tr>
<tr>
<td>$N_{\text{col,com}}^c$</td>
<td>Capacity design-derived column compression force</td>
</tr>
<tr>
<td>$N_{\text{col,i,com}}^c$</td>
<td>Capacity design-derived compression force in a column at level i</td>
</tr>
<tr>
<td>$N_{\text{col,i,ten}}^c$</td>
<td>Capacity design-derived tension force in a column at level i</td>
</tr>
<tr>
<td>$N_{\text{brace}}^c$</td>
<td>Overstrength compression capacity of a brace</td>
</tr>
<tr>
<td>$N_{\text{brace,i}}^c$</td>
<td>Overstrength compression capacity of a brace at level i</td>
</tr>
<tr>
<td>$N^*$</td>
<td>Design axial force (tension or compression)</td>
</tr>
<tr>
<td>$N_{\text{beam}}^*$</td>
<td>Design axial force in a beam</td>
</tr>
<tr>
<td>$N_{\text{brace}}$</td>
<td>Design axial force in a brace</td>
</tr>
<tr>
<td>$N_{\text{col,com}}^*$</td>
<td>Design compression force in a column</td>
</tr>
<tr>
<td>$r$</td>
<td>Radius of gyration</td>
</tr>
<tr>
<td>$T$</td>
<td>Translational period of vibration</td>
</tr>
<tr>
<td>$T_1$</td>
<td>Fundamental (translational) period of vibration, for direction under consideration</td>
</tr>
<tr>
<td>$V_{\text{b,vcr}}$</td>
<td>Capacity design-derived average brace vertical component</td>
</tr>
<tr>
<td>$V_{\text{beam,G+Q}}$</td>
<td>Shear force in a beam due to long term gravity load combinations G &amp; Q_u</td>
</tr>
<tr>
<td>$V_{\text{beam,GQ_u}}$</td>
<td>Same as above</td>
</tr>
<tr>
<td>$V_{\text{fn}}$</td>
<td>Nominal shear force of bolt</td>
</tr>
<tr>
<td>$V_{\text{beam}}^c$</td>
<td>Capacity design-derived beam shear force</td>
</tr>
<tr>
<td>$V_E^c$</td>
<td>Collector beam design shear force</td>
</tr>
<tr>
<td>$V_{\text{beam}}^*$</td>
<td>Design shear force in a beam</td>
</tr>
<tr>
<td>$V_{\text{total,brace,com}}^*$</td>
<td>Vertical component of compression force from brace into column</td>
</tr>
<tr>
<td>$w_i$</td>
<td>Section weight per metre length (kg/m) at level i</td>
</tr>
<tr>
<td>$w_G$</td>
<td>Uniformly distributed dead load (kN/m)</td>
</tr>
<tr>
<td>$w_{Q_u}$</td>
<td>Uniformly distributed long-term live load (kN/m)</td>
</tr>
<tr>
<td>$Z$</td>
<td>Hazard factor</td>
</tr>
<tr>
<td>$\alpha_c'$</td>
<td>Compression member slenderness reduction factor</td>
</tr>
<tr>
<td>$\gamma_E$</td>
<td>Upper limit seismic actions factor (see NZS 3404, Clause 12.9.1.3)</td>
</tr>
<tr>
<td>$\delta_{ui}$</td>
<td>The inter-storey displacement for storey i for the ultimate limit state as specified in Clause 7.3.1 of NZS1170.5</td>
</tr>
<tr>
<td>$\Delta_{b,max}$</td>
<td>Maximum brace displacement</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$\theta_c$</td>
<td>Acute angle between a brace in compression and the collector beam at the top end of the brace</td>
</tr>
<tr>
<td>$\theta_t$</td>
<td>Acute angle between a brace in tension and the collector beam at the top end of the brace</td>
</tr>
<tr>
<td>$\Theta$</td>
<td>Stability coefficient</td>
</tr>
<tr>
<td>$\mu$</td>
<td>Structural ductility factor</td>
</tr>
<tr>
<td>$\varphi$</td>
<td>Strength reduction factor (see NZS3404 Table 3.3)</td>
</tr>
<tr>
<td>$\varphi_{om}$</td>
<td>Overstrength factor incorporating only material variation</td>
</tr>
<tr>
<td>$\varphi_{oms}$</td>
<td>Overstrength factor incorporating statistical variation in yield stress and an allowance for strength increase due to strain-hardening</td>
</tr>
</tbody>
</table>
Chapter 1
INTRODUCTION

1.1 Background
New Zealand (NZ) is located at a moderate to high seismic prone area. Recently, Christchurch and its surrounding area had been devastated by a series of earthquakes starting from December last year at 2010, with the most severe earthquake recorded at February 22, 2011 with a magnitude of 6.3 on the Richter Scale, followed by thousands of aftershocks. This earthquake caused significant damage and failure of unreinforced masonry buildings and damage, and limited failure to reinforced concrete buildings. On the other hand, steel structures located in Christchurch have performed beyond expectations considering the severity of the earthquake, most notably structures with a seismic resisting system such as Eccentrically Braced Frames (EBF) or Concentrically Braced Frames (CBF) (Bruneau et al, 2011). However, repair of this braced structure, designed and built to conventional procedures costs time and impacts on the economy of the recovering cities. Figure 1.1 shows an example of the repairs needed following the most severe of the earthquake series, on 22 February 2011.

![Fractured EBF (left) and Buckled CBF (right) (Bruneau et al., 2011)](image)

With the increased recognition of the post-earthquake economic impacts on society has come the increased demand for seismic resisting systems that will deliver a high damage threshold in severe earthquakes, allowing buildings to be rapidly returned to service, requiring little or
no structural repair. This civil engineering research focuses on the development of one such system. It is known as Buckling Restrained Braces (BRB) and has been applied in both North America (Figure 1.2) and Japan, recognised for its superior seismic performance compared to existing braced systems due to its suppression of brace compression buckling, as the braces in a BRB system do not buckle in compression. They have similar strength and stiffness in tension and compression which makes design of the structural system easier. However, in North America and Japan, they have been implemented into the market as proprietary, patent protected items for use in conjunction with the relevant national based design procedure. This does not suit their application in New Zealand. The concept of the BRB system itself is not proprietary, but the configuration and details of the brace assembly is commonly subjected to US patent laws (Hussain et al., 2005).

![Figure 1.2: Examples of BRBs system at UC Berkeley](image)

The BRB systems are more sustainable compared to other braced frames as they achieve relatively greater strength with less material in comparison, allowing construction with less overall steel. The increased compression capacity of the BRB system also lowers and simplifies the foundation and connection design requirements. (Snoonian, 2005)

To enable the usage of the BRB system commercially in NZ, a reliable generic design method must be developed from an existing design method and incorporated in compliance to NZ standards and CBF design procedures. To achieve this, a method has been developed and used to design a standard multi-storey building as a study case. An example brace and bay from
that study case has then been physically built and experimentally tested on in accordance with the established North American experimental testing regime. These tests performed well and as a result, recommendations have been able to be made for the brace and system designs.

1.2 Objectives

The principal objective of this research has been to establish, through experimental testing and numerical modelling, the adequacy of a proposed design method for the generic design of a BRB system in compliance with New Zealand (NZ) Standards. This was achieved by:

1. Development of a design procedure and application to a 10 storey building located in Christchurch with compliance to NZ standards and CBF design provisions
2. Development of a design method and detailing requirements for the brace and connections, and application of these to the construction of a test frame and two braces, one with pinned end connections and the other with bolted gusset plate end connections
3. Simulating seismic loading of these braces to determine their behaviour
4. Development of a numerical model and application of numerical integration time history analysis to determine the system performance (this section was limited due to time constraints placed on the master’s project).

The overall objective has been to undertake the necessary research from which to develop a generic design and detailing guide for BRB application in New Zealand.

1.3 Research Methodology and Timeline

The main aim of this research has been to develop a robust design guide and detailing for a Buckling Restrained Braces system that allows it to be widely applied in New Zealand.

The BRB concept and design guides that have been developed for North America, with the aid of proprietary systems, are available. Furthermore, Alistair Fussell of SCNZ has written general guidelines for designing BRB in NZ (Fussell, 2010b)

The following steps have been implemented:

1. A study case design of a multi-storey building with the BRB system in accordance with the braced frame design provisions modified to account for the BRB system
Chapter 1: Introduction

2. Development of generic design and detailing requirements for the BRB system in accordance to NZ Standards
3. Production of construction drawings for the BRB (frame and braces) system to allow a frame to be constructed by a steel fabricator. Documentation of the BRB construction process covering the steps required to manufacture the generic braces and their connections into the frame
4. Performing quasi-static and seismic-dynamic sub-assemblage tests to determine the performance of the BRB system developed under inelastic cyclic loading
5. Performing time history analyses with computer software to determine the performance of the BRB system
6. Developing the final design and detailed recommendations.

<table>
<thead>
<tr>
<th>Time Period</th>
<th>Activities</th>
<th>Deliverables</th>
</tr>
</thead>
<tbody>
<tr>
<td>March 2011 – April 2011</td>
<td>Literature Review, developing the methodology and revision on NZ standard on multi-storey design</td>
<td>Establishment of methodology and development of preliminary design steps</td>
</tr>
<tr>
<td>April 2011 – May 2011</td>
<td>Design of the study case multi-storey building</td>
<td>Design recommendations for BRB system in accordance to NZ standards</td>
</tr>
<tr>
<td>June 2011 – July 2011</td>
<td>Detailing of the system and construction drawing for the fabricator</td>
<td>Detailing recommendations for BRB system and construction drawings for the specimen</td>
</tr>
<tr>
<td>July 2011 – August 2011</td>
<td>Supervision of the specimen construction, experimental testing programme established and learning Ruaumoko</td>
<td>Documentation of the construction process and experimental testing programme</td>
</tr>
<tr>
<td>September 2011</td>
<td>Transport and installation of specimen and BRB model on Ruaumoko</td>
<td>Representative BRB model for time history analysis</td>
</tr>
</tbody>
</table>
1.4 Outline of Thesis

The thesis is organised into 7 chapters:

- **Introduction** – Chapter 1 introduces the background of the project and the methodology used for this research.

- **Literature review** – Chapter 2 investigates the history and development of the BRB system and their configurations, including their design philosophy and testing protocol. A discussion regarding the experimental tests and analytical modelling of the BRB system is also included.

- **Proposed Design of BRB** – Chapter 3 demonstrates a step-by-step design method which is used to design the study case building in compliance to NZ standards, as well as design recommendations.

- **Detailing and Construction of BRB specimen** – Chapter 4 outlines the design method which is used to design the connections for the sub-assemblage specimen. The construction details and specifications of the specimen are also provided.

- **Experimental Testing** – Chapter 5 investigates the performance of the BRB system when subjected to dynamic sub-assemblage tests. The testing procedures and setup are provided. An analysis of results and discussion are also included.

- **Time History Analysis** – Chapter 6 reports the results and analyses of an initial 2D RUAUMOKO model of the sub-assemblage specimen when subjected to an earthquake.

- **Conclusions** – Chapter 7 provides a summary of research findings and recommendations in future work in this area.
Chapter 2

LITERATURE REVIEW

This chapter outlines the literature review conducted for this research project. The knowledge detailed in this section was used to define the scope of the project and to form the generic design procedure proposed in this thesis.

2.1 Introduction to Literature Review

The literature review is organised into five different sections. The concept and theory behind the BRB is presented in Section 2.2. This section reveals the purpose behind the BRB system and explains the advantages and disadvantages of the system compared to conventional concentrically braced frames.

This is followed by the development of BRB in Section 2.3. In this section, the early conception in Japan to the implementation in the United States of the BRB system is presented. This section also shows the original model of the system and its development to the modern generic system implemented in recent times.

Several other reviews of analytical and testing studies which have been used to test the performance of the BRB system are summarised in Section 2.4.

The studies on the configuration and design of BRB and their findings are summarised in Section 2.5. This part of the literature interview is crucial in forming the proposed generic design procedures which are presented in chapters 3 and 4 of this thesis.

Section 2.6 presents the summary of the literature review.
2.2 Concept of BRB

2.2.1 Conception
When a building is subjected to an earthquake, a significant amount of kinetic energy is distributed into the structure and the level of damage sustained by the building depends on the dissipation of this energy. Earthquake resisting systems are designed to dissipate energy efficiently from the structure by yielding at controlled locations and in a ductile manner. A braced frame system is one such earthquake resisting system (Ko and Field).

![Figure 2.1: Behaviour of conventional brace and BRB (Xie, 2005)](image)

The braced frame system is designed to resist structural frame distortions when subjected to lateral forces. In a severe earthquake, the braces are subjected to extreme loading with repeated cycle of stress, which exceed the elastic limits of the brace. The braces will then yield in compression and tension to absorb and prevent the build-up of energy in the structure. The bracing ability of conventional steel, in this case, is limited by its tendency to buckle due to the combination of compression force and the unbraced lengths of the steel core, which results in unsymmetrical hysteretic behaviour as shown in Figure 2.1. After buckling occurs, the ability of the braced member to resist earthquake loading and to dissipate energy will be severely reduced. This also leads to complex design methods as the behaviour of a buckled brace is very unpredictable. Therefore, research has been focused towards finding a way to prevent the buckling of the brace, and to ensure that the same strength can be achieved by the brace in both tension and compression, which simplifies the design process.
The BRB system is designed to suppress this undesirable mode of failure by providing continuous lateral restraint (Figure 2.2), which ensures that the brace length of the core elements is effectively zero. Therefore, the BRB system develops a balanced hysteresis loop when subjected to cyclic loading, with its compression yielding response similar to its tension yielding response (Fussell, 2010a). This behaviour is the trademark of the BRB system and is the distinguishing feature as compared to other conventional braces in terms of hysteretic response (Figure 2.1).

![Figure 2.2: Mechanics of BRB (Lopez & Sabelli, 2004)](image)

Due to the ability of the BRB system to restrain buckling and its symmetrical hysteretic response, the system clearly has the advantage in performance compared to conventional CBF. The full, stable hysteretic response also means less redistribution of loads and deformations in BRB compared to conventional braces. The ease the designers have in manipulating the area of steel core allows them to make the capacity of each storey closer to the demand easily and thus mitigates the tendency of damage concentration in weak stories. In addition, as the braces do not buckle laterally, damage on adjacent non-structural elements can be reduced (Sabelli et al., 2003).

The behaviour of the BRB system does, however, have several disadvantages. The BRB system has a relatively low post-yield stiffness which may still lead to the concentration of damage in one level, even though the brace capacities can be balanced throughout different storeys. There is also a difference in the tensile and compressive capacities of the brace, which raises issues in the design of V-braced configurations. Lastly, the ability to specify the strength of the system to as close as the design force as possible may reduce the inherent
overstrength of the overall system, leading to slightly increased ductility demand; however, this would be a minor effect.

2.2.2 Anatomy of a Buckling Restrained Braces System

![Diagram of BRB element minus connection and projected section](image)

Figure 2.3: Anatomy of BRB element minus connection and projected section (Sabelli et al., 2003)

The following is a brief explanation on the anatomy of a widely used configuration of the BRB system. At the present state, the BRB system is comprised of 4 parts (Iano, 2005) as shown in Figure 2.3:

1. An inner yielding steel core, being the primary load bearing component of the system which resists both tension and compression axial forces subjected to the system by lateral forces. This is where the yielding takes place.
2. Mortar, used as filler material between the steel tube and steel core to resist buckling stress. Other filler material such as reinforced concrete and grout can also be used as filler material.
3. De-bonding material, an important part of the system which separates the core and the filler, thus allowing the core to freely move and expand due to tension and shorten due to compression within the mortar.
4. A surrounding steel tube, enveloping the inner member and restraining the steel core from buckling.
2.3 Development of BRB

2.3.1 Timeline

<table>
<thead>
<tr>
<th>Event</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Invention</td>
<td>Early 1980’s</td>
</tr>
<tr>
<td>Testing</td>
<td>Mid 1980’s</td>
</tr>
<tr>
<td>Implementation in Japan</td>
<td>February 1988</td>
</tr>
<tr>
<td>Technology transfer to US</td>
<td>1998</td>
</tr>
<tr>
<td>US Testing / Simulation</td>
<td>Spring 1999</td>
</tr>
<tr>
<td>Implementation in United States</td>
<td>January 2000</td>
</tr>
</tbody>
</table>

Figure 2.4: Timeline of BRB system development (Ko and Claire)

2.3.2 Japan

The initial concept of BRB was first developed in Japan. The system was initially designed where flat steel braces were sandwiched between precast concrete panels to provide effective lateral restraint (Wakabayashi et al., 1973). In the 1980s, the effort to eliminate the buckling failure mode of slender elements led to a collaborative effort between Professor Wada of the Tokyo Institute of Technology with Nippon Steel Construction, resulting in the current model of the BRB system. The design was inspired by a typical human bone which is bigger at the end with a reduced section in the middle (Hussain et al., 2005). Further research found that the de-bonding process is very important to enable the brace to resist horizontal loading, while the concrete panel only serves to prevent the brace form buckling. The results also showed that in order to allow deformation of the stiffened ends in the precast panels, it is important to provide gaps between the exposed and embedded parts. Figure 2.5 shows the test setup and results to determine the performance of the designed system (Wakabayashi et al., 1973).

Figure 2.5: BRB test setup and its hysteresis (Wakabayashi et al., 1973)
Kimura et al. in 1976 performed the first test on steel braces encased in filler mortar steel tubes without the de-bonding agent, while Fujimoto et al. in 1988 extended this research by testing it with the de-bonding agent (Xie, 2005). The BRB system is then first used in Japan for its energy dissipation mechanism in conjunction with moment braced frames in 1988 (Hussain et al., 2005), and by 2000 the BRB system is the most widely used type of damper in high-rise buildings constructed in Japan.

2.3.3 United States
In 1999, the first test of the BRB system was conducted at UC Berkeley. The tests conducted demonstrated good performance of the BRB system under various loading histories. In 2000, the first BRB system is applied in North America as a primary lateral resisting system at UC Davis (Hussain et al., 2005).

At 2004, full-scale testing of Special Conventional Braced Frames (SCBF) by UC Berkeley demonstrated poor inelastic performance due to inherent buckling behaviour. In contrast, the laboratory results showed that the BRB frame system demonstrated a superior performance compared to the SCBF system as shown in Figure 2.6. The tests also revealed that failures of the system were mostly located in supplementary elements of the BRB frame, which in turn, contributed to the out-of-plane buckling of the connection. (Hussain et al., 2005)

![Figure 2.6: BRB test setup and its hysteresis response (Hussain et al., 2005)](image-url)
2.3.4 New Zealand
The first application of BRB in New Zealand was in 1991, used in the extensions to the Department of Geography at the University of Canterbury. These braces were developed from first principles using the tube, mortar and Denso tape concept tested in this project.

2.4 Design and Testing Requirement

2.4.1 United States Requirement
Currently, BRB system design in the United States is governed by the 2003 NEHRP Recommended Provisions for New Buildings and Other Structures (FEMA 450) and the 2005 AISC Seismic Provisions for Structural Steel Buildings. Furthermore, the Structural Engineers Association of Northern California (SEAONC) has established a document (SEAONC, 2001) on recommended provisions for the BRB system which contains the testing requirement for a BRB system to be acceptable for use. The loading sequence contained in this document, reproduced below, has been used by many engineers to determine whether the performance of their BRB system is adequate to fulfil the acceptance criteria set by AISC:

1. 6 cycles of loading at the deformation corresponding to \( D_b = D_{by} \)
2. 4 cycles of loading at the deformation corresponding to \( D_b = 0.50 \ D_{bm} \)
3. 4 cycles of loading at the deformation corresponding to \( D_b = 1 \ D_{bm} \)
4. 2 cycles of loading at the deformation corresponding to \( D_b = 1.5 \ D_{bm} \)
5. Additional complete cycles of loading at the deformation corresponding to \( D_b = 1 \ D_{bm} \) as required for the Brace Test Specimen to achieve a cumulative inelastic axial deformation of at least 140 times the yield deformation (not required for the Sub-assemblage Test Specimen)

Symbol Definitions

\( D_b \): Deformation quantity used to control loading of the Test Specimen (total brace end rotation for the Sub-assemblage Test Specimen; total brace axial deformation for the Brace Test Specimen)

\( D_{bm} \): Value of deformation quantity, \( D_b \), corresponding to the Design Storey Drift

\( D_{by} \): Value of deformation quantity, \( D_b \), at first significant yield of Test Specimen

A comprehensive design guide on seismic design of the BRB system has also been developed by Lopez and Sabelli (2004) in conjunction with the 2005 AISC Seismic Provisions for
Structural Steel Buildings. In addition, Choi and Kim (2006) proposed a BRB design method based on energy requirements, while Maley et al. (2010) developed a displacement-based design method for dual systems with BRB and moment-resisting frames.

2.4.2 New Zealand Requirement

To the best of the author’s knowledge, there are no established requirements or appropriate guides on designing BRBs in New Zealand, which was the principal motivation for undertaking this project. Despite this, several papers have been published in New Zealand regarding the system based on overseas research and design procedures. Fussell (2010a) provides an overview of the BRB system, reviewing its concept, development and seismic performance, but more importantly, Fussell (2010b) provides a short design guide for the BRB system. This design guide, however, does not prove its reliability through experimental testing or analytical modelling and does not include design examples.

2.5 Other Analytical and Experimental Testing Studies

2.5.1 Experimental Testing

There have been many studies done to confirm the behaviour and performance of the BRB system; however, large scale BRB frame tests have shown poor performance. Several of these studies, such as that conducted by Roeder et al. (2006), found that gusset plate distortion and brace instability, considered as an undesirable mode of failure, has been observed as a storey drifts between 0.02 and 0.025 radians.

Prominent and recent large-scale tests were conducted by Fahnestock et al. (2007). These experimental evaluations were conducted to demonstrate the performance of the system when subjected to multiple earthquake simulations and investigated the poor performance at storey drifts between 0.02 and 0.025 radians. Figure 2.7(a) shows the large-scale experimental test setup of Fahnestock et al.

These experimental tests have demonstrated that a well detailed BRB frame system can withstand significant drifts and ductility demands by BRB with no significant damage. The improved connection details, and in particular the stocky gusset plates, pinned connections and end collars of the BRB seen in Figure 2.7(b), were considered as the primary reasons that
Chapter 2: Literature Review

the system avoids the undesirable mode of failure which had plagued previous BRB experimental tests. Unfortunately, the design of this pinned collar is not included in the published paper.

![Large scale test setup and beam-column-brace connection details](image)

**Figure 2.7:** (a) Large scale test setup and (b) beam-column-brace connection details (Fahnestock et al., 2007)

Research on new BRB configurations is still on-going with the aim to improve the performance of the system and eliminate its disadvantages. Examples of new BRB configurations include the hybrid buckling restrained braced frames (Atlayan & Charney, 2012), which help to prevent excessive damage on the system under frequent low-to-mid intensity ground motions, and the Self-Centring BRB (Miller & Fahnestock, 2012), incorporating a NiTi shape memory which helps in self-centring. Other uses of BRB are also explored, such as the research conducted by Dinu et al. (2012) on the performance of a non-seismic reinforced concrete frame which has been strengthened by BRB.

### 2.5.2 Analytical Modelling

Aside from experimental tests, rigorous analytical modelling has also been conducted to determine the performance of the BRB system. In 2003, a comprehensive analytical study was conducted by Sabelli et al. (2003) to determine the seismic response of three and six storey CBF buildings utilising BRB. The research concludes that the BRB managed to overcome problems usually associated with the special and conventional types of CBF.
To accurately determine the performance of the BRB system, the analytical model results should approximate the experimental test results. Tremblay et al. (2004) performed both the experimental tests and analytical modelling, and a comparison is made between both results, shown in Figure 2.8. The comparison concludes that the model is reasonably accurate. The analysis of the BRB was performed with RUAUMOKO using the Ramberg-Osgood hysteretic model.

![Graph showing experimental testing and analytical modelling results](image)

**Figure 2.8: Experimental testing and analytical modelling results (Tremblay et al., 2004)**

Analytical modelling was also used by Asgarian and Amirhesari (2008) to compare the performance of BRB and ordinary braced frames. The research concludes that the BRB has a better performance after comparisons were made in terms of storey drifts, storey shears, storey shear versus drift hysteresis behaviour and plastic hinge locations.

### 2.6 Configuration of BRB

As mentioned in Chapter 1, the concept of BRB itself is not proprietary; however, the configuration of the system and details of the brace assembly is commonly subject to US Patents. The following details a literature review from published papers regarding research on the recommended configuration and detailing of BRB.

#### 2.6.1 Bracing Configuration

Several different bracing configurations can be considered when designing the BRB system. Deulkar et al. (2010) used five different configurations for their study shown in Figure 2.9 on
the BRB system to help with vibration control. The projects compared the reduction in roof displacements (Figure 2.10) obtained from analyses of different bracing configurations and found that the inverted V-bracing has the least roof displacement of the tested configurations.

![Figure 2.9: Experimental testing and analytical modelling results (Deulkar et al., 2010)](image)

![Figure 2.10: Comparison of displacement between different types of BRB bracing configurations (Deulkar et al., 2010)](image)

2.6.2 De-bonding Agent

Various de-bonding agents have been employed in the course of BRB research. The first research conducted on BRB de-bonding material is detailed in Wakabayashi et al. (1973) where, possible de-bonding materials such as epoxy resin, silicon resin and vinyl tapes were tested; before it was decided to use a coating of silicon resin layer on top of epoxy resin as a composite de-bonding material. Other de-bonding materials such as silicon painting, styrol foam, polyethene film sheets, and silicon rubber sheets were also used by other studies (Xie, 2005). However, there is no indication on the best-performing de-bonding agent.

2.6.3 Filler Material

Early studies of the BRB have used reinforced concrete or mortar as a filler material to restrain the buckling of the steel brace. A study performed by Gheidi et al. (2009) investigates
the effect of filler material. In their research, uniaxial tests were conducted to three specimens of BRB with different filler materials. The results are shown in Figure 2.11, where filler material of specimen (a) is normal concrete, specimen (b) is aggregates and specimen (c) is lean concrete.

![Figure 2.11: Comparison of hysteresis result between different types of filler material (Gheidi et al., 2010)](image)

From the results, it can be concluded that normal concrete performs the best compared to the other filler materials. The research had also found that 25-30 MPa concrete was adequate in preventing local and global buckling of the flat plates.

Several studies also construct their specimens without any filler material; however, considering the risk and uncertainties in constructing a seismic resisting element and the availability of filler materials, it is safer to utilise filler material in the design of BRB (Gheidi et al., 2009).

### 2.6.4 Steel Tube

The ability of the BRB elements to resist flexural buckling can be limited by the thickness of the encasing or steel tube. This, in turn, also affects the local buckling of the core plate and thus the entire strength and stability of the system. In the study conducted by Takeuchi et al. (2010), various arrangements of the tube restrainer were subjected to cyclic loading tests and numerical analyses to investigate the influence of the restrainer tube on the system
performance. The research found that the local buckling failure on the plate may happen, depending on the ratio of width- or diameter-to-thickness of the rectangular or circular tube. A higher ratio, in this case 65 for the rectangular tube, shows evidence of local buckling failure when tested, while a ratio of 25 shows no evidence of local buckling failure.

Research conducted by Ju et al. (2009) also shows that BRBs with 4 mm and 5 mm thick encasing tubes have a 34-54% higher compressive strength compared to BRBs with a 3 mm thick encasing tube. This is shown in Figure 2.12.

![Figure 2.12: Comparison of hysteresis results between BRBs with different encasing tube thickness (Young et al., 2009)](image)

2.6.5 Shapes of BRB
Xie (2005) has compiled various cross-sectional geometries that have been used for studies of the BRB system, shown in Figure 2.13. Like the de-bonding agent, there is no certainty about the best performing cross-section; although the most commonly used cross-sections are the cruciform and flat plate shapes.
2.6.6 Gusset Plate

There were several studies performed to determine the design requirements of the gusset plates. Wigle and Fahnestock (2009) recommends that the gusset plates should have sufficient thickness to prevent large distributed stresses, but stiff enough to prevent large stress concentrations that may cause out-of-plane buckling. It was also recommended to make the gusset plate as compact and efficient as possible, as stronger gusset plates require thicker welds and have less stiffness and ductility required for a seismic resisting system.

2.6.7 End Connection

Hussain et al. (2005) found that there are three common configurations for BRB end connections. These connections have been developed by three major manufacturers of BRB. One configuration, manufactured by Nippon Steel, has a typical standard bolted connection, while CoreBrace has developed a modified bolted connection. Another BRB manufacturer, Star Seismic, has developed a true pinned connection. The advantages and disadvantages of each type of connection are listed in Table 2.1 and shown in Figure 2.14.
Table 2.1: Different types of BRB Connection (Xie, 2005)

<table>
<thead>
<tr>
<th>Type of Connection</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
</table>
| Standard Bolted   | • Bigger holes allow more erection tolerance compared to pinned connections  
• Multiple bolts provides more redundancy  
• Better distribution of forces to the gusset plate  
• As it is not a true pinned connection, secondary moment or overturning moment forms between the connection and brace  
• High installation cost due to the number of bolts and splices  
• Larger gusset plates to accommodate the bolts, and hence shorter core length, resulting in larger strains compared to pinned connections | |
| Modified Bolted   | • Same as standard bolted  
• No splices and significantly fewer bolts  
• Same as standard bolted | |
| True Pin          | • Longer BRB core length resulting in smaller strains for a given load  
• Eliminates overturning moment  
• Less installation cost  
• Smaller erection tolerance | |
Tsai from the National Taiwan University has also developed double-Tee double-tube connections. The connection shown in Figure 2.15 has the advantage of requiring only one set of bolts for each brace ends, resulting in a reduced length of connection compared to the traditional braced connection (Xie, 2005).

Figure 2.14: Standard bolted connection; (b) modified bolted connection, and (c) true pinned connection (Xie, 2005)

Figure 2.15: Proposed configurations of BRB connections (Xie, 2005)
2.7 Summary of Literature Review

A summary of the literature review is presented below:

- The concept of the BRB system is simple and well understood; however, there are many configurations and methods by which to construct the system.
- The reliability and the performance of the BRB and its superior seismic response compared to other bracing systems has been confirmed by many experimental tests and analytical modelling.
- The primary elements of the BRB system have been kept the same, while recent tests focused on the improving the performance of the system by determining its optimal configuration.
- New Zealand has little exposure in the design and application of BRB and no official provisions regarding the design of BRB systems to the best of the author’s knowledge.
In order to fulfil the first objective of this research project, a design procedure was proposed and then implemented for the design of a multi-storey building with BRBs as its primary seismic resisting system. The design procedure outlined in this chapter is the procedure followed by the author before additional recommendations and some changes were made after the installation and experimental testing of the specimen. These changes do not affect the overall system design procedure but are more concerned around the design and detailing of the BRB and its connections.

3.1 Introduction

The proposed BRB design procedure is based on the concentrically braced frame (CBF) design procedure contained in HERA Report R4-76 (1995), with modifications to account for BRB design requirements. This report is readily available in New Zealand and had been used for many projects by NZ structural engineers, which will help the reliability and spread of the proposed design procedure.

The static design force for all structural members is determined by a static equilibrium approach, involving either the equivalent static method or the modal response spectrum method. For this project, the equivalent static method of NZS 1170 (2005) Clause 6.2 was used. This is a force based design method.

The BRB design requirements contained in this chapter also consider all the recommendations by Fussell (2010b) on the design of BRBs.

Full details of the design example are found in Appendix A, with this chapter giving a summary of the procedure.

This chapter is comprised of the following:
Section 3.2 describes the study case building configuration used as the base of the design example and the procedure performed by the author in this research project.

The first part of Section 3.3 presents the preliminary design procedure. This section contains a derivation of the equivalent static forces and design actions in accordance with NZS 1170.5 (2005). It also outlines the method used to check the earthquake-induced deflections of the structure.

The second part of Section 3.3 provides the design procedure used to determine the structural member size and specific design requirements for BRB. The detailing requirements are covered in the next chapter.

Section 3.4 summarises the output of the design procedure and provides a comparison of structural member sizes across different options in the design procedure.

### 3.2 Study Case Building

The study case building plan layout, shown in Figure 3.1, has been taken from “The Fundamentals of Seismic Design”, a SESOC seminar at June 2008, for a fully working preliminary design example of a three storey Eccentrically Braced Frame (EBF) structure (SESOC & IStrucE, 2008). The layout has also been used for several University of Auckland research projects on multi-storey design, for example Patel (2009), which allows the results of different research projects to be compared and assists in checking for accuracy.

The 10 storey building is 36 m high (Figure 3.2) and is symmetrical about both principal plan axes. Along each axis an inverted V-braced BRB structural system is utilised for resistance to lateral forces; and therefore each frame will resist half of the seismic actions. The striped line represents the BRB position in the structure. The design undertaken in this project is for braces in the east to west direction.

The structure is composed entirely of steel frames with composite floor slabs. The design of the composite floor slabs and gravity supporting columns is not considered in this thesis.
The Christchurch earthquake series has provided a comprehensive set of data that can be used in time history analyses of the designed structure, despite the work being outside the scope of this project. Hence, the study case building is located at Christchurch with a soft or deep (Class D) soil type and hazard factor (Z) of 0.22 in accordance with NZ 1170.5 (2005).

The study case structure is an office-type building. Therefore, it has an importance level of 2 according to NZS 1170.5 and is designed for a working life of 50 years.

As shown in Figure 3.2, the inverted V-braced BRB configuration has been chosen for the study case. Time history analyses of different configurations performed by Deulkar et al. (2010) found that the inverted V-braced type BRB configuration produces the least amount of roof displacement compared to other types of configurations. Thus, the configuration has been chosen to provide good control in lateral displacement of the structure. It also suits the bay dimension (width and height) and is a common layout for CBF and eccentrically braced frame buildings.

The BRB arrangement is symmetrical to fulfil the condition required in the preliminary design of CBF stated in clause 12.12.2.2 of NZS3404, where the forces between braces should not differ by 20%.
3.3 Design Procedure

The design procedure is composed of two parts. The first part deals with the preliminary design of the structure, while the second part focuses on the design of the structural members.

The BRB system can be considered as a fully ductile system or a system with limited ductility. Thus, the system is classified as a category 1 or category 2 structure according to the draft of NZS3404.7 (2011), as shown in Table 3.1.

<table>
<thead>
<tr>
<th>Category of Structure</th>
<th>Category of Member</th>
<th>Braces</th>
<th>Collector Beams</th>
<th>Columns</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>3</td>
<td>2</td>
<td>2</td>
</tr>
</tbody>
</table>

The design procedure will closely follow the methodology to design category 1 and category 2 V-braced CBFs with braces effective in compression and tension contained in HERA report R4-76 Section 17.

3.3.1 Preliminary Design

**Step 1**

The first step to design a braced frame is to check whether a maximum height limitation is required and, if so, whether it is satisfied. The height limitation for CBFs is stated in NZS3404 Clause 12.12.4.1 and depends on the brace slenderness ratio. The brace slenderness value is given by the following formula.

\[
\frac{k_{e}L}{r} \sqrt{\frac{f_{y}}{250}}
\]

Normal brace systems fail under compression loading by member compression buckling due to the slenderness ratio of the braces. The yielding steel core used in the BRBs is encased by mortar and a confining tube which effectively provides continuous lateral restraint and suppresses member buckling. Thus, it can be considered as a system with a compression brace slenderness ratio ≤ 30 when implementing the NZS 3404 provisions. Furthermore, because
the inelastic strength and stiffness in tension and compression are identical, the height limits for category 3 responses can apply to a category 1 or 2 system (advice from Associate Professor Charles Clifton). Therefore, the height limitations of Table 12.12.4.(3) of NZS3404 apply and allow a V-braced system of up to 12 storeys. The design example building complies with these recommendations.

**Step 2**
The factor $C_s$ found in C12.12.3 of NZS3404 accounts for inelastic behaviour of a CBF system, recognising that this behaviour is less stable and dependable than for an eccentrically braced frame or moment resisting frame system. The value is derived from the following three variables:

1. The variable $A$ accounts for the deterioration in inelastic performance of CBF with increasing brace slenderness. The expression for the variable is shown by Equation 3.2. In this case, because the BRB suppresses member buckling, the post-buckling compression capacity of BRB is taken as 1. Therefore, the variable $A$ will be taken as 1 when designing the BRB system.

$$A = 1/[0.5(1 + \alpha_c)]$$  \hspace{1cm} (3.2)

2. The variable $B$ accounts for the departure of the CBF system from the optimum weak beam strong column (overall) mechanism towards the less desirable strong beam – weak column (shear) mechanism. In NZS3404 it is set as a function of the height of the building compared with the maximum height limit set by the relevant information from Table 12.12.4, ranging from 1.1 for structures under 1/3 of this limit to 1/3 for structures over 2/3 of this limit. On that basis the value of $B$ for this design example would be 1.3. However, it can be argued that a well-designed and detailed BRB with capacity designed collector beams and columns will develop a dependable overall mechanism as an eccentrically braced frame which does not require this magnification. In this example, the latter argument has been adopted to give $B = 1$.

3. Lastly, the variable $C$ accounts for the influence of inelastic demand on the system. The variables are found by modifying values of products taken from variables $A$ and $B$. With $A$ and $B$ both equal to 1, $C_s = 1$, which is the value used in this design.
Step 3

In this step, the different load cases and combinations are considered. Loads applied during the preliminary design are shown in Table 3.2. These loads are used to calculate the seismic weight of the building and assess the lateral deflection of the structure. The loads are mostly consistent with loads used in SESOC seminar at June 2008 mentioned earlier.

<table>
<thead>
<tr>
<th>Load</th>
<th>Pressure (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Self-Weight (Frame only)</td>
<td>0.6</td>
</tr>
<tr>
<td>Superimposed Dead Load (SDL)</td>
<td>0.5</td>
</tr>
<tr>
<td>Self-Weight of Floor</td>
<td>3.15</td>
</tr>
<tr>
<td>Live Load on Floor (LL)</td>
<td>3</td>
</tr>
<tr>
<td>Live Load on Roof</td>
<td>0.25</td>
</tr>
<tr>
<td>Cladding</td>
<td>1</td>
</tr>
</tbody>
</table>

Consideration of $E_{\text{max}}$ will not be required as the $C_s$ value is taken to be 1.0 and the core of the brace, which is the primary seismic-resisting system, can be very closely matched to the demand. Thus, the overstrength factor of the system will be close to the minimum possible and hence the capacity design derived design actions will be less than the upper limit design actions. In this case, $E_{\text{max}}$ is three times $E$ according to HERA Report R4-76. For other scenarios, however, if the $C_s$ value is not 1 or varies from building heights the case must be considered using Equation 3.3.

$$\gamma_E = \frac{C_h(T_1, 1)C_{s,l}}{C_h(T_1, \mu)C_s}$$ (3.3)

Initially, a time period is calculated using Equation 3.4 from NZS1170.5 Commentary Clause C4.1.2.2.

$$T = 1.25 \times k_t \times h_n^{0.75}$$ (3.4)

The design earthquake actions are obtained using the equivalent static method from NZS1170.5 (2005). To calculate the elastic site hazard spectrum, $Z$ has been taken as 0.22 which is the old value before changes were made in early 2011. This is done to ensure that time history analysis results on the system’s performance using the Christchurch earthquake acceleration data can be compared to the performance of other existing buildings in Christchurch which have been subjected to the same earthquake loading.
A structural system ductility factor of 3 has been chosen for the design example. This is less than the maximum structural ductility factor of 4 recommended by Fussell (2010b). The design procedure also incorporates design actions determined from structural ductility factors of 2.4 and 2 for the preliminary sizing of beams and columns, respectively. The lesser values of ductility are chosen to anticipate the increase in size of the member due to the overstrength factor required for the capacity design procedure. All steel material, unless stated, have a nominal yield strength, $f_y$, of 300 MPa.

After the horizontal seismic shears with different ductility factors are calculated for one of the frames in one direction, the various beams, braces and columns can be sized. Detailed methods on sizing these elements will be included later in this section. The recommended procedure (Clifton, 2010) of deciding which groups of storeys will have the same member strength (in this case BRB yielding core strength), determining the average seismic design actions over each group and matching the design capacity to this average design action will be followed. The sections were split into 3-3-4, meaning that the top three storeys have the same design force which had been averaged from the initial design force from the top three storeys. The next three storeys and the last four storeys are also designed by considering the average design force of the corresponding sections. The design values are found in Table 3.3.

### Table 3.3: Preliminary Design Force (kN)

<table>
<thead>
<tr>
<th>Level</th>
<th>Brace ($\mu = 3.0$)</th>
<th>Beam ($\mu = 2.4$)</th>
<th>Column ($\mu = 2.0$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Initial Force</td>
<td>Avg Force</td>
<td>Initial Force</td>
</tr>
<tr>
<td>R</td>
<td>159.58</td>
<td>258.91</td>
<td>199.47</td>
</tr>
<tr>
<td>9</td>
<td>262.73</td>
<td></td>
<td>323.64</td>
</tr>
<tr>
<td>8</td>
<td>354.42</td>
<td></td>
<td>430.03</td>
</tr>
<tr>
<td>7</td>
<td>434.66</td>
<td>499.60</td>
<td>543.32</td>
</tr>
<tr>
<td>6</td>
<td>503.43</td>
<td>629.28</td>
<td>624.51</td>
</tr>
<tr>
<td>5</td>
<td>560.73</td>
<td>700.92</td>
<td>741.10</td>
</tr>
<tr>
<td>4</td>
<td>606.58</td>
<td>646.70</td>
<td>758.22</td>
</tr>
<tr>
<td>3</td>
<td>640.96</td>
<td></td>
<td>801.21</td>
</tr>
<tr>
<td>2</td>
<td>663.89</td>
<td></td>
<td>829.86</td>
</tr>
<tr>
<td>1</td>
<td>675.35</td>
<td></td>
<td>844.19</td>
</tr>
</tbody>
</table>
Step 4
A structural analysis is performed in this step to assess the P-Delta Effects and to check the seismic lateral deflections. The program used for this analysis is SAP2000. A 2D layout of the building is constructed in the computer program, shown in Figure 3.3.

![Figure 3.3: Model Elevation](image)

Several modifications have been made to the layout to improve the structure model. The elastic stiffness of the steel beams has been increased by a factor of 1.2 in accordance with NZS3404 Appendix N Clause N1.1.2(a)(i). A rigid zone factor of 0.5 had been utilised to take account for the stiffening of the member in the joint zone between structural members. Offsets have been placed to represent where connection centres do not meet the corner of the beam and column joints. All offset calculations have been based at half the depth of the beam.

The floor slabs provide high in-plane stiffness and rigidly tie the columns and floor system beams together at each level. To take account of this behaviour, a rigid diaphragm has been assigned to each floor, which will ensure that each node on a level experiences the same lateral deflection.

Rotational springs have been used in the model to better represent the fixed base connection of the inner columns and pinned base connection of the outer columns. Both connections, in
reality, will never be fully rotational, or infinitely rotationally stiff. The springs will be used to take account of these factors and produce a more realistic value to the base connection stiffness. The formulae for a fixed base connection (Equation 3.5) and for a pinned connection (Equation 3.6) are listed below (Patel, 2009).

\[
K_\theta = \frac{1.67EI_c}{L_c} \quad (3.5)
\]

\[
K_\theta = \frac{0.1EI_c}{L_c} \quad (3.6)
\]

The analysis is performed on one frame, which will take account for half of the earthquake design lateral force. Each gravity outer column of the model has been modified to take account of the contribution from the stiffness of the other columns, as they play an important role in accurately determining the seismically induced deflection. The seismic weights are calculated to account for half of the building and are assigned to the two outer nodes of the storeys.

The BRB elements are connected to the steel frame by a gusset plate connection. This type of connection has been modelled as providing a pinned restraint from the brace to the surrounding frame; this is an approximation as the connection is somewhere between fixed and pinned under in-plane rotation and closer to pinned for out-of-plane rotation. The beams and columns, in addition to the gusset plate connection, are connected by the moment endplate which has been modelled as providing a fixed connection between the two elements. Therefore, the model will specify the brace connection to the frame as pinned and the beam to column connection as fixed. This is a reasonable representation of the overall stiffness of the beam/brace/column connection.

The BRB elements can be separated into the yielding and non-yielding elements. When subjected to inelastic demand, the inelastic deformation on the BRB elements is principally limited to the yielding region of the steel core. Therefore, it is important to model the BRB as yielding length and not work-to-work point length as shown in Figure 3.4. A ratio in the order of 0.5 to 0.7 for the range of yielding length to work point length and a ratio of 2.0 to 1.4 to correspond to the effective stiffness of a braced element with the length ratio are recommended (Fussell, 2010b).
This is quite a range in recommended stiffness parameters and, to determine the most appropriate values to use in modelling the BRB elements, several variations on these parameters in the study case building have been utilised. These have been based on the preliminary design of the study case building which is located at Auckland and not Christchurch. The first model listed in Table 3.4 is the 1.5 $A_e$, where the BRB elements have been constructed as a continuous member with the area of the steel core multiplied by 1.5. The second model is segmented, where each BRB element has been split into three sections with a length ratio of 0.2:0.6:0.2 for non-yielding: yielding: non-yielding components. All models have been given realistic offsets to locate where the brace element will start and end.

The simply supported and propped cantilevers are design options for the collector beams that can be chosen and their significance will be discussed further along the chapter. The steel core cross-section used in the analyses is a cruciform shape before it was changed to a rectangle in the detailing.

Table 3.4 contains the result of static analyses performed when the models are subjected to the lateral forces obtained by the equivalent static method. All deflection results are in millimetres while the periods are in seconds. The seismic weights, as mass sources, have also been incorporated for these analyses and the periods are obtained by the Rayleigh method from NZS1170.5 (2005).

The design procedure in this chapter assumes that the building centre of rigidity is located at its centre of mass. Therefore, accidental eccentricity and torsional analysis contained in section 6 of NZS 1170.5 (2005) does not need to be considered in the lateral force calculation.
Table 3.4: Summary of different model options analysis (mm)

<table>
<thead>
<tr>
<th>Level</th>
<th>Simply Supported</th>
<th></th>
<th>Propped Cantilever</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.5 $A_s$</td>
<td>Segmented</td>
<td>1.5 $A_s$</td>
<td>Segmented</td>
</tr>
<tr>
<td>R</td>
<td>4.55</td>
<td>5.1</td>
<td>4.4</td>
<td>5</td>
</tr>
<tr>
<td>9</td>
<td>6.04</td>
<td>6.7</td>
<td>5.6</td>
<td>6.3</td>
</tr>
<tr>
<td>8</td>
<td>5.7</td>
<td>6.3</td>
<td>5.3</td>
<td>6</td>
</tr>
<tr>
<td>7</td>
<td>4.55</td>
<td>5.1</td>
<td>4.4</td>
<td>5</td>
</tr>
<tr>
<td>6</td>
<td>5.94</td>
<td>6.7</td>
<td>5.7</td>
<td>6.5</td>
</tr>
<tr>
<td>5</td>
<td>7.2</td>
<td>8.3</td>
<td>6.9</td>
<td>7.9</td>
</tr>
<tr>
<td>4</td>
<td>7.6</td>
<td>8.7</td>
<td>7.2</td>
<td>8.4</td>
</tr>
<tr>
<td>3</td>
<td>7.3</td>
<td>8.4</td>
<td>7</td>
<td>8.1</td>
</tr>
<tr>
<td>2</td>
<td>6.6</td>
<td>7.6</td>
<td>6.3</td>
<td>7.3</td>
</tr>
<tr>
<td>1</td>
<td>4.6</td>
<td>5.1</td>
<td>4.4</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>T = 0.79 s</td>
<td>T = 0.84 s</td>
<td>T = 0.77 s</td>
<td>T = 0.83 s</td>
</tr>
</tbody>
</table>

As observed from Table 3.4, the difference between the deflections results from the segmented model and 1.5 $A_s$ model peaks at approximately 1.1 mm. The 1.5 $A_s$ model achieved a slightly lesser deflection compared to the segmented model. The period resulting from the analysis of the 1.5 $A_s$ model is lower than that of the segmented model, which in turn results on a greater final design force. Therefore, the 1.5 $A_s$ model will be adopted in this design procedure as there is a negligible difference between the deflection of both models and this option provides a slightly more conservative approach to obtaining the final design force.

After the study case model is constructed, static analyses are performed to find the deflection resulted from the lateral forces. From the deflection, the fundamental period of the building can be calculated using the Rayleigh method stated in section 5 of NZS1170.5 (2005). The fundamental period is then used to calculate a revised lateral force coefficient, new member sizes and a new fundamental period. The process is iterative and performed until the previous fundamental period differs by less than 10% of the new fundamental period.
Chapter 3: Proposed Design Procedure of BRB

Figure 3.5: Mode 1 (left), $T = 2.45$ s and mode 2 (right), $T = 0.83$ s of the study case building analysis result

Figure 3.6: Final design lateral force (left) and deflected shape (right) of the study case building analysis result
The fundamental period of the study case building is 2.45 seconds. It is noted that the analysis resulted in a period which is considerably higher than that first obtained from Equation 3.4. However, studies on EBF systems performed by SCNZ using the computer program ETABs over the years had found that the empirical equations given in the loading standards with building height as a variable underestimate the fundamental period (SCNZ, 2011).

The graph shown in Figure 3.7 summarises the finding of the SCNZ results. The blue line represents the fundamental period obtained by the analysis of the ETABs model, while the pink and green lines represent the period found by using the formula contained in NZS1170 and the uniform building code of the United States, respectively.

![Graph showing comparison of fundamental period obtained by standard empirical formulae and analysis results (SCNZ, 2011)](image)

Figure 3.7: Comparison of fundamental period obtained by standard empirical formulae and analysis results (SCNZ, 2011)

Considering the line of best fit of the research results, the expected fundamental period of the study case building is 2.02 seconds. This meant that the fundamental period found in this study case is still higher than expected. A small part of the reason for this is the allowance for realistic flexibility in the column base to foundation connection. A larger contribution is the flexibility of the braces, due to the very small size of their yielding core compared with that of an EBF brace. This means a structure with BRBs is more flexible than the brace/beam system for an EBF with short braces.
The earthquake-induced deflection check follows the steps contained in section 7 of the NZS1170.5 (2005). In addition to the displacement found from the analysis, the standard also considers the increase in displacements due to P-delta effects. These effects, however, are not required in this study case in accordance to clause 6.5.2(c) of section 6 of the structural design action standards (NZS1170.5, 2005). The analysis found that the stability coefficients calculated with Equation 3.7 are less than 0.1 for all the storeys of the study case building.

\[
\theta = \frac{W_i \delta_{ui}}{V_i (h_i - h_{i-1})}
\]  

(3.7)

The displacements shown in the table below have been calculated in accordance with NZS1170.5 (2005) Clause 7.3.1 and incorporating the scaling factor found in clause 6.2.3. The displacements are then checked by both of the displacement profiles suggested by clause 7.2.1.1 with the inter-storey deflection limit stated in clause 7.5.1 of NZS1170.5 (2005). The limit states that the inter storey deflections shall not exceed 2.5% of the corresponding storey height. As shown in Table 3.5 the study case building drift ratio satisfies the limit, which is expected due to the inherent stiffness of the BRB system.

<table>
<thead>
<tr>
<th>Level</th>
<th>Deflection (m)</th>
<th>Drift Ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>0.09</td>
<td>0.68</td>
</tr>
<tr>
<td>9</td>
<td>0.08</td>
<td>0.80</td>
</tr>
<tr>
<td>8</td>
<td>0.07</td>
<td>0.88</td>
</tr>
<tr>
<td>7</td>
<td>0.06</td>
<td>0.82</td>
</tr>
<tr>
<td>6</td>
<td>0.05</td>
<td>0.83</td>
</tr>
<tr>
<td>5</td>
<td>0.04</td>
<td>0.83</td>
</tr>
<tr>
<td>4</td>
<td>0.03</td>
<td>0.73</td>
</tr>
<tr>
<td>3</td>
<td>0.02</td>
<td>0.77</td>
</tr>
<tr>
<td>2</td>
<td>0.01</td>
<td>0.57</td>
</tr>
<tr>
<td>1</td>
<td>0.01</td>
<td>0.48</td>
</tr>
</tbody>
</table>

### 3.3.2 Structural Member Design

**Step 5**

This step of the design procedure deals with the design of the brace member. After the design force has been determined, a check should be made to ensure that the forces sustained by each brace on each storey do not differ by more than 20%. In this study case, due to symmetrical arrangement of the braces, this is satisfied.
In the design standard of HERA report R4-76, the gravity loading supported by the collector beam should be considered in designing the brace compression capacity in addition to the lateral loading. This reflects on the Equation 3.8 which requires the design force for the brace to include the load combination of G & Q_u contained in section 17 of the HERA report R4-76.

\[ N_{\text{brace}}^* = N_{\text{brace,E}} + N_{\text{brace,GQ_u}} \quad (3.8) \]

For the study case, the design procedure has considered two options for the design of the system, as shown in Figure 3.8. The first option is the simply supported design where the beams will be designed to resist the full force of gravity loading as a continuous member without any support from the brace. The second option is the propped cantilever design. This option designs the beams as two separate members supported by the brace at the brace-to-beam intersection.

![Simply supported](Brace does not take gravity loading)

![Propped Cantilever](Brace takes gravity loading)

*Figure 3.8: Comparison between a simply supported and propped cantilever design*

Both options have been developed in the study case design. The simply supported design option yields a much larger beam compared to the propped cantilever design. The bending moment resisted by the simply supported beams is 4 times larger than that resisted by the propped cantilever beams. In turn, however, the simply supported option yields a smaller brace size compared to the propped cantilever due to the absence of additional axial load from gravity loading. A summary of the structural members is presented in Section 3.4 to compare the output of both design options.
The materials used to design the brace will have an $f_y$ of 300 MPa which fulfils the requirement for category 2 braces contained in NZS3404 (2009). The slenderness ratio requirements which are used to take account of brace post-buckling under compression can be ignored in the BRB design as this mode of brace buckling is suppressed. The elements of the BRB are singly symmetric, thus satisfying the geometric requirements.

The steel core area of the brace shown in section C-C (Figure 3.9) is rectangular in these designs. The capacity of the steel core area can be easily determined by the formula shown in Equation 3.9 and requires the compression and tension capacity to resist the entire axial force running through the brace. Due to the ability of the BRB to suppress buckling, the tension and compression capacity of the steel core will be equal. A strength reduction factor of 0.9 is required by NZS3404 (1997).

$$\Phi N_c = \Phi N_t = \Phi f_y A_{sc} \quad (3.9)$$

In the same Figure, the non-yielding elements shown throughout section A-A and B-B are designed by the same formula, but must resist the design action from the core after the overstrength factor is applied.

Fussell (2010b) recommends making a check on the ratio of the design action on the brace and its capacity. The ratios should be balanced and similar over the height of the structure in order to help suppress a concentration of inelastic action at one level. The averaging procedure for determining the BRB core areas described in Section 3.3.1 step 3 will assist in keeping these ratios balanced. It is a significant advantage of BRBs over conventional CBFs that this close matching of design capacity to design demand can be made.
The design of the steel tube which surrounds the mortar and steel core has been fully adopted from Fussell’s paper (2010b) on the design of BRB. The method determines the stiffness required to be possessed by the casing to prevent the steel core from buckling. This check must be made for both in-plane and out-of-plane buckling if the casing is not doubly symmetrical. The formula shown in Equation 3.10 is derived from the relationship between the Euler buckling load of the casing and the nominal section capacity of the brace. The formula will calculate the minimum moment of inertia required for the casing. For the full derivation see the referenced paper. The bending capacity of the steel tube must also be able to resist bending forces generated by a transverse load equivalent to 2.5% of the brace axial force applied as a point load in the middle of its tube (Equation 3.11).

\[ I_{\text{min}} = \frac{1.5L^2f_yA_s}{\pi^2E} \]  

(3.10)

Note that, following the testing of the two connection systems, it is recommended that the \( I_{\text{min}} \) for a pinned brace is increased by 1.25 times the value from Equation 3.10. For further details, see Section 5.8.

\[ \Phi M_{\text{casing}} > 0.025\Phi_{\text{oms}}f_yA_s \]  

(3.11)

For the study case building the steel tube sizes have been chosen from the list of available sizes of square hollow sections (SHS) on design capacity tables for structural steel hollow sections (ASI, 2004). However, the study case found that the size of the steel tubes most likely depends on the size of non-yielding elements due to detailing requirements.

Step 6

This step implements the capacity design procedures. In this study case, the yielding element of the BRB has been specified as the weakest link in the structural system. Following the Capacity Design concept, the yielding element must behave in a ductile manner and the strength of the structure depends on the steel core area size. All other structural members are designed for the Capacity Design derived design actions from the steel core to ensure that the chosen ductile failure mechanism develops. Allowance should also be made to account for greater strength of the material than specified. Therefore, the strength of other structural members in the system is required to be greater than the maximum action that can be resisted by the steel core, taking into account a higher specified strength of material and increase in strength due to strain hardening.
The overstrength ($\Phi_{oms}$) used to determine the size of non-yielding elements is 1.25. This value corresponds to a category 1 member and steel grade of 300 MPa from Australia and New Zealand and is taken from Table 12.2.8(1) of NZS3404 (1997). The rest of the design procedure which incorporates the brace overstrength compression capacity ($N_{brace}^{oc}$) can be calculated according to Equation 3.12. To be conservative, $\Phi_{om}$ will be taken as 1.3, on the basis that the steel is not supplied from either Australia or New Zealand.

\[ N_{brace}^{oc} = 0.9\Phi_{om}N_c \]  
(3.12)

**Step 7**

The collector beam capacity design derived actions are determined at this stage of the design procedure. HERA report R4-76 first considers the net vertical seismic force acting on the collector beams, in step 7.1, section 17.2. These downward acting forces, in an inverted V braced system, are applied to brace/collector beam joints due to the imbalance between the post-buckled brace compression capacity and the brace tension capacity. As the BRB system supresses brace buckling in compression, the net vertical seismic force can be considered negligible. This is further supported by the formula used to calculate the forces shown in Equation 3.13, where the member slenderness reduction factor ($\alpha_c$) can be taken as 1 for the BRB system. Given that the BRB is set up symmetrically, the out-of-balance force would only be at most approximately 10% of the axial force.

\[ F_{beam,i}^c = (1 - \alpha_c)N_{brace,i}^{oc}\sin(\theta_c) \]  
(3.13)

The next step in the design process is to determine the collector beam design bending moments and axial forces. Pin end support will be assumed at beam ends to simplify the design bending moment calculation. HERA report R4-76 recommends that the collector beam spans between column centrelines due to the effect of compression of brace buckling. However, the BRB system has been designed to supress buckling, and thus it is plausible for the collector beam to be supported vertically by the brace. In step 5, design options for the collector beam are introduced and, depending on which option is chosen, different design bending moments will be obtained.

The following formulae, reproduced from HERA report R4-76, are used to calculate the design bending moments. For the BRB system $F_{beam}^c$ will be 0 in Equation 3.14, so the design bending moments are only calculated from gravity loading (Equation 3.15). The $L$ in Equation 3.15 varies depending on which design option is chosen.
Chapter 3: Proposed Design Procedure of BRB

\[ M_{beam}^* = 0.5F_{beam}^c L' + M_{beam, GQ_u} \]  
\[ M_{beam, GQ_u}^* = \frac{(W_G + W_{Q_u})L^2}{8} \]

The collector beams on each level should also be designed to resist the axial force. The design axial force originates from the transfer of the horizontal component of the brace force from the storey above to the storey below. Therefore, the design axial force of the collector beams will be derived from the maximum axial forces resisted by the steel core with an overstrength factor as shown in Equation 3.16. Figure 3.10 can be used to discern the appropriate level to be considered in the equation for the inverted V-braced configuration.

\[ N_{beam} = N_{beam, i} = N_{brace, i}^c \left( \frac{\sin(\theta_c)}{\tan(\theta_c)} \right) \]

Upon completion of the analysis, it was found that designing the beam for the axial loads from analysis using \( \mu = 2.4 \) to find the preliminary design axial forces generates a slightly understrength beam when it is treated as simply supported and hence the gravity loading does not contribute to additional design axial forces to the brace. However, designing the beam as a propped cantilever and accounting for the axial load from gravity loading caused the axial load design to be severely underestimated when compared to the initial axial force found when \( \mu = 2.4 \). This is due to the axial force of the collector beams being dependent on the
overstrength axial forces of the braces and the propped cantilever design option has a stronger brace compared to the simply supported option.

<table>
<thead>
<tr>
<th>Floor</th>
<th>N* Beam Propped Cantilever</th>
<th>N* Beam Simply Supported</th>
<th>N*beam at μ = 2.4</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>353.7</td>
<td>188.3</td>
<td>161.8</td>
</tr>
<tr>
<td>9</td>
<td>353.7</td>
<td>188.3</td>
<td>161.8</td>
</tr>
<tr>
<td>8</td>
<td>353.7</td>
<td>188.3</td>
<td>161.8</td>
</tr>
<tr>
<td>7</td>
<td>542.6</td>
<td>342.2</td>
<td>312.3</td>
</tr>
<tr>
<td>6</td>
<td>542.6</td>
<td>342.2</td>
<td>312.3</td>
</tr>
<tr>
<td>5</td>
<td>542.6</td>
<td>342.2</td>
<td>312.3</td>
</tr>
<tr>
<td>4</td>
<td>601.0</td>
<td>457.0</td>
<td>404.2</td>
</tr>
<tr>
<td>3</td>
<td>601.0</td>
<td>457.0</td>
<td>404.2</td>
</tr>
<tr>
<td>2</td>
<td>601.0</td>
<td>457.0</td>
<td>404.2</td>
</tr>
<tr>
<td>1</td>
<td>601.0</td>
<td>457.0</td>
<td>404.2</td>
</tr>
</tbody>
</table>

This design procedure strongly recommends the use of the simply supported design option. Considering the beam as simply supported throughout the whole length is the most conservative way to estimate the design bending moments. Despite providing a larger beam, the simply supported design option will provide smaller braces which reduces the overstrength design action for the column and connection of the system. The smaller braces are also easier to detail and will be shown in Chapter 4. Stronger and stiffer collector beams, in conjunction with semi-rigid bolted connections to the columns, also contribute to the self-centring ability of the braces. The larger beam is less likely to deform under severe loading and provides a stronger anchor for the braces to return to its original position compared to the smaller beam. Therefore, the simply supported design option is considered to perform better in earthquakes. Time history analyses will be required to confirm this and this is one of the future research recommendations made.

**Step 8**

After the design actions of the beams are determined in the previous step, the beams will be designed for the capacity design derived bending moment, axial force and combined actions.
The beams will be designed in accordance with sections 5, 6 and 8 of NZS3404 (2009) using the Capacity Design. The designs of the beams are most likely to be governed by the combined actions of axial forces and bending moments. The collector beams must also satisfy the special requirements of Clause 12.12.2.5 in NZS3404. The other requirements stated in the HERA report R4-76 design guide should also be satisfied where applicable.

The beam capacity design derived shear force is calculated from Equation 3.17 in accordance to the HERA report R4-76. In this study case design, it is assumed that the plastic hinge will form at the critical section on each side of the beam-brace connection, as shown in Figure 3.11. The $L'$ used for this study case is 3.5 m.

$$V_{beam}^{c} = \frac{1}{L'} \left( \phi_{om} M_{rx} - M_{beam,cQ_{u}} \right) \quad (3.17)$$

![Figure 3.11: Distance of $L'$ and $L$ and typical beam-brace connection for inverted V-braced configuration (HERA report R4-76, 1995)](image)

An equation is also given to limit the beam capacity design derived shear force in the form of Equation 3.18. This equation, however, may not be suitable for this design procedure as values $N_{c,brace}$ and $N_{t,brace}$ of the BRB element are almost equal. Given that the braces are arranged symmetrically and $\alpha'_{c}$ is 1 and $\phi_{om}$ is 1.3, the limit is 0.155 kN.

$$V_{beam}^{c} \leq \frac{b}{L} \left[ \phi_{om} N_{t,brace} \sin(\theta_{c}) - \alpha'_{c} \phi N_{c,brace} \sin(\theta_{c}) \right] \quad (3.18)$$

Fussell recommends taking the collector beam shear force as shown in Equation 3.19. This formula takes into account the frame action caused by the in-plane rigidity of the brace/beam/column end plate connection.
The following table shows the difference between the beam seismic shear force results depending on the method used to calculate them. The final design force will then be taken as the sum of the beam seismic shear force and shear force due to long term gravity loading as shown in Equation 3.20.

\[ V^*_{\text{beam}} = V_{\text{beam}} + V_{\text{beam, GQ}} \geq V_{\text{beam, GQ}} \]  \hspace{1cm} (3.20)

**Table 3.7: Summary of beam seismic shear forces in kN**

<table>
<thead>
<tr>
<th>Floor</th>
<th>( V^c_{E} ) (Fussell) Propped Cantilever</th>
<th>( V^c_{E} ) (R4-76) Propped Cantilever</th>
<th>( V^c_{E} ) (Fussell) Simply Supported</th>
<th>( V^c_{E} ) (R4-76) Simply Supported</th>
<th>( V_{\text{Beam G+Q}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>14.0</td>
<td>10.4</td>
<td>45.4</td>
<td>41.2</td>
<td>86.6</td>
</tr>
<tr>
<td>9</td>
<td>14.0</td>
<td>10.4</td>
<td>45.4</td>
<td>41.2</td>
<td>118.0</td>
</tr>
<tr>
<td>8</td>
<td>14.0</td>
<td>10.4</td>
<td>45.4</td>
<td>41.2</td>
<td>118.0</td>
</tr>
<tr>
<td>7</td>
<td>21.4</td>
<td>8.8</td>
<td>51.2</td>
<td>49.9</td>
<td>118.0</td>
</tr>
<tr>
<td>6</td>
<td>21.4</td>
<td>8.8</td>
<td>51.2</td>
<td>49.9</td>
<td>118.0</td>
</tr>
<tr>
<td>5</td>
<td>21.4</td>
<td>8.8</td>
<td>51.2</td>
<td>49.9</td>
<td>118.0</td>
</tr>
<tr>
<td>4</td>
<td>25.1</td>
<td>8.1</td>
<td>51.2</td>
<td>42.3</td>
<td>118.0</td>
</tr>
<tr>
<td>3</td>
<td>25.1</td>
<td>8.1</td>
<td>51.2</td>
<td>42.3</td>
<td>118.0</td>
</tr>
<tr>
<td>2</td>
<td>25.1</td>
<td>8.1</td>
<td>51.2</td>
<td>42.3</td>
<td>118.0</td>
</tr>
<tr>
<td>1</td>
<td>25.1</td>
<td>8.1</td>
<td>51.2</td>
<td>42.3</td>
<td>118.0</td>
</tr>
</tbody>
</table>

The study case design uses the method suggested by HERA report R4-76 to find the critical design shear. In addition, the collector beam shear capacity check can be ignored as plastic hinges should not form in BRB systems. However, it is unlikely that the collector beam shear capacity influences the design of the collector beam.
Step 9

The column seismic axial forces of the BRB system can be calculated by considering case I, step 9.1.1, section 17.1 of HERA Report R4-76 (1995), where the braces do not actually buckle. The forces can be calculated by Equation 3.21, and involves the overstrength compression capacity of the brace being used in column design at the lowest brace level affecting the column being designed, with the design compression capacity being used above this. The nominal compression capacity is the yield capacity of the core, $A_s f_y$. The formula presented here refers to a three storey building with an inverted V-braced configuration, where $i$ is used to represent the first storey.

$$
\left( N_{\text{col},i,\text{ten}}^c \right)_{\text{case1}} = \left( N_{\text{col},i,\text{com}}^c \right)_{\text{case1}}
$$

$$
= \left( N_{\text{brace}}^c \sin(\theta_\text{c}) \right)_{i+1} + \sum_{i+2}^{n} \left( \varphi N_{\text{c,brace}}^c \sin(\theta_\text{c}) \right)
$$

(3.21)

The column design axial forces can then be found by summing the appropriate compression forces due to gravity loads and column design derived from Equation 3.22.

$$
N_{\text{col,com}}^* = N_{\text{col,GQa}} + N_{\text{col,com}}^c
$$

(3.22)

The BRB elements will not buckle, and therefore consideration of increasing the gravity load by including the vertical component of the brace is irrelevant.

Fussell (2010b) has suggested the use of Equation 3.23 to calculate the seismic shear force. The formula presented here refers to a four storey building with an inverted V-braced configuration, with the first storey designated as $i$. For further information about the formula, refer to Fussell’s (2010b) paper on the design of BRB. In this formula $V_{b,\text{vcr}}$ is the capacity design derived average brace vertical component and $V_{c,\text{vcr}}$ is the capacity design derived collector beam seismic shear, which is not accounted for in the R4-76 formula.

$$
N_{ci}^c = \sum_{i+1}^{i+3} V_{b,\text{vcr}} + \sum_{i}^{i+3} V_{c,\text{vcr}}
$$

(3.23)
Table 3.8: Summary of column seismic axial forces in kN

<table>
<thead>
<tr>
<th>Floor</th>
<th>( N_{\text{cot}}^C ) (Fussell) Propped Cantilever</th>
<th>( N_{\text{cot}}^C ) (R4-76) Propped Cantilever</th>
<th>( N_{\text{cot}}^C ) (Fussell) Simply Supported</th>
<th>( N_{\text{cot}}^C ) (R4-76) Simply Supported</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>45.4</td>
<td>0.0</td>
<td>45.38</td>
<td>0.0</td>
</tr>
<tr>
<td>9</td>
<td>409.2</td>
<td>318.5</td>
<td>260.32</td>
<td>169.6</td>
</tr>
<tr>
<td>8</td>
<td>699.6</td>
<td>563.4</td>
<td>436.12</td>
<td>300.0</td>
</tr>
<tr>
<td>7</td>
<td>995.7</td>
<td>808.4</td>
<td>617.75</td>
<td>430.4</td>
</tr>
<tr>
<td>6</td>
<td>1461.9</td>
<td>1223.4</td>
<td>937.93</td>
<td>699.4</td>
</tr>
<tr>
<td>5</td>
<td>1888.9</td>
<td>1599.2</td>
<td>1226.14</td>
<td>936.5</td>
</tr>
<tr>
<td>4</td>
<td>2315.8</td>
<td>1974.9</td>
<td>1514.35</td>
<td>1173.5</td>
</tr>
<tr>
<td>3</td>
<td>2795.4</td>
<td>2403.3</td>
<td>1905.90</td>
<td>1513.8</td>
</tr>
<tr>
<td>2</td>
<td>3262.8</td>
<td>2819.6</td>
<td>2273.60</td>
<td>1830.4</td>
</tr>
<tr>
<td>1</td>
<td>3730.2</td>
<td>3235.8</td>
<td>2641.30</td>
<td>2146.9</td>
</tr>
</tbody>
</table>

As observed from Table 3.7, Fussell’s approach yielded higher axial forces when compared to the HERA report R4-76 due to the addition of the beam shear force. The design procedure adopts the approach based on the HERA report R4-76 as the BRB system is not expected to generate a downward-acting seismic shear component.

This design procedure recommends that the column design bending moments are found using the method recommended by the HERA report R4-76, where the eccentricity of the applied resultant axial force from the collector beam and brace is considered along with collector beam shear forces to the column centrelines. Note that in the design example this procedure has not been followed and the design bending moments are obtained from the SAP2000 analyses, as the design moments are usually very small.

**Step 10**

The columns will be designed in accordance to the relevant sections of NZS3404 (2009) as stated in the R4-76. The design of the columns is most likely governed by the combined actions of axial forces and bending moments. The other requirements stated in the HERA report R4-76 design guide should also be satisfied where applicable and other load combinations considered.

The design procedure for detailing is included in Chapter 4.
3.4 Design Summary Output

The following tables contain the member sizes obtained from the proposed design procedure. These are determined before any detailing requirements are considered and therefore should not be considered the final member sizes for the study case structure.

*Note that the shape of the steel cores have been taken as cruciform in this study case before they are changed into rectangular in the detailing phase of the project*

3.4.1 Simply Supported Design Output

<table>
<thead>
<tr>
<th>Level</th>
<th>Collector beams</th>
<th>Columns</th>
</tr>
</thead>
<tbody>
<tr>
<td>8th FL. – Roof Fl.</td>
<td>360UB50.7</td>
<td>200UC52.2</td>
</tr>
<tr>
<td>4th FL. – 7th FL.</td>
<td>360UB56.7</td>
<td>310UC96.8</td>
</tr>
<tr>
<td>Gr. FL. – 3rd FL.</td>
<td>360UB56.7</td>
<td>350WC197</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Yielding Segment</th>
<th>Tube (C350)</th>
<th>First Plate</th>
<th>Second Plate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Section Size</td>
<td>Length</td>
<td>Width</td>
</tr>
<tr>
<td>8th FL. – Roof Fl.</td>
<td>100 x 100 x 3.0</td>
<td>46</td>
<td>12</td>
</tr>
<tr>
<td>4th FL. – 7th FL.</td>
<td>125 x 125 x 4.0</td>
<td>64</td>
<td>16</td>
</tr>
<tr>
<td>Gr. FL. – 3rd FL.</td>
<td>150 x 150 x 3.0</td>
<td>72</td>
<td>20</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Non-Yielding Segment</th>
<th>First Plate</th>
<th>Second Plate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Length</td>
<td>Width</td>
</tr>
<tr>
<td>8th FL. – Roof Fl.</td>
<td>92</td>
<td>12</td>
</tr>
<tr>
<td>4th FL. – 7th FL.</td>
<td>116</td>
<td>16</td>
</tr>
<tr>
<td>8th FL. – Roof Fl.</td>
<td>142</td>
<td>20</td>
</tr>
</tbody>
</table>
### 3.4.2 Propped Cantilever Design Output

**Table 3.11: Propped Cantilever – Beam and Column Sizes**

<table>
<thead>
<tr>
<th></th>
<th>Girder beams</th>
<th>Columns</th>
</tr>
</thead>
<tbody>
<tr>
<td>8th FL. – Roof Fl.</td>
<td>250UB25,7</td>
<td>200UC59.5</td>
</tr>
<tr>
<td>4th FL. – 7th Fl.</td>
<td>250UB31.4</td>
<td>310UC118</td>
</tr>
<tr>
<td>Gr. FL. – 3rd Fl.</td>
<td>310UB32.0</td>
<td>350WC230</td>
</tr>
</tbody>
</table>

**Table 3.12: Propped Cantilever – BRB Size (Cruciform Shape in mm)**

<table>
<thead>
<tr>
<th>Yielding Segment</th>
<th>Tube (C350)</th>
<th>First Plate</th>
<th>Second Plate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Section Size</td>
<td>Length</td>
<td>Width</td>
</tr>
<tr>
<td>8th FL. – Roof Fl.</td>
<td>125 x 125 x 4.0</td>
<td>66</td>
<td>16</td>
</tr>
<tr>
<td>4th FL. – 7th Fl.</td>
<td>125 x 125 x 4.0</td>
<td>80</td>
<td>20</td>
</tr>
<tr>
<td>Gr. FL. – 3rd Fl.</td>
<td>150 x 150 x 3.0</td>
<td>88</td>
<td>20</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Non-Yielding Segment</th>
<th>First Plate</th>
<th>Second Plate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Length</td>
<td>Width</td>
</tr>
<tr>
<td>8th FL. – Roof Fl.</td>
<td>116</td>
<td>16</td>
</tr>
<tr>
<td>4th FL. – 7th Fl.</td>
<td>118</td>
<td>20</td>
</tr>
<tr>
<td>8th FL. – Roof Fl.</td>
<td>144</td>
<td>20</td>
</tr>
</tbody>
</table>
3.4.3 Summary
This chapter has shown that HERA Report R4-76 (1995/2001) section 17, regarding the Design Procedures for V-braced CBF with braces effective in compression and tension, can be readily converted into a BRB design guide with several modifications. The single most important modification is to understand that the BRB system is able to suppress buckling. This causes the system to be subjected to less design requirements required by the CBF system.

The BRB system is laterally stiff, meaning that design lateral deflections are low. This is clearly seen by the drift ratio obtained from the analysis which does not exceed 1%, and thus is nowhere near the 2.5% limit. It is prudent to check the displacement after the member sizes have been sized according to both the design and detailing requirements; however, this is unlikely to be a problem due to the stiffness of the braced system.

As observed from Tables 3.9-3.12, the simply supported design option resulted in larger collector beam sizes when compared to the propped cantilever design option. However, the propped cantilever has larger steel core areas and in turn larger non-yielding element sizes. The difference between column sizes is almost negligible, although the propped cantilever columns are slightly larger. The proposed design procedure recommends using the simply supported design option.

In the next section the detailing requirements for the brace and gusset plate are implemented. Caution should be taken as these may lead to changes in member sizes.
Chapter 4

DETAILING AND CONSTRUCTION OF THE BRB TEST SPECIMENS

This chapter presents the process followed to construct the sub-assemblage specimens used for experimental testing. A detailed explanation of this process is important as there is a lack of information regarding the design and detailing requirements of the system, especially the brace itself and the connections to the frame. One of the principal objectives of this project is to develop generic details, which are presented in this chapter.

4.1 Introduction

The clearance requirements of the BRB system are developed from the expected deformation of the yielding length. The connection details are mostly derived from HERA Design Construction and Bulletin (DCB) No. 56 (HERA, 2000), 61 (HERA, 2001a), and 63 (HERA, 2001b).

The sub-assemblage specimens are designed for braces located between levels 7 and 8 of the study case building. The collector beam designs are based on the simply supported design as it yields a smaller brace for ease of construction and smaller force requirement for testing, while delivering a larger collector beam size which, in conjunction with the bolted semi-rigid connections, will enhance the tendency to self-centre, following severe shaking. The force that can be applied for experimental testing is limited to the strength of the University of Auckland’s dynamic actuator, which is 300 kN. A smaller brace will also have an increased chance of failure under loading compared to a bigger brace, as the capacity of the smaller brace is influenced more by the friction between its surface area and the restraining elements. The overall slenderness ratio is also higher for a smaller brace. This is because, as the size of the yielding core of the brace gets larger, there is not much increase in its perimeter surface area compared to the increase of its cross-sectional area, and thus the smaller the brace, the more it will be influenced by frictional forces and potentially weakened by design of overlong or poorly restrained braces.
The detailing requirements outlined in this chapter have been used to design the sub-assemblage specimens and design examples can be found in Appendix B. The procedure explained in this chapter incorporates several figures presented in the design example to illustrate the detailing check performed in this procedure.

This chapter is organised as follows:

Section 4.2 contains all the detailing requirements considered when the sub-assemblage specimens were designed. This section is split into two parts. The first section details the connection design procedure of the whole system and the second section deals with BRB-specific detailing requirements. Complete assembly and construction drawings are given in Appendix C.

Section 4.3 presents the method used to find the clearance in the detailing of the BRB system. This section also presents the use of SAP2000 to estimate the displacement of the sub-assemblage system.

Section 4.4 contains recommendations on choosing the materials used for specimen construction and details the properties on the chosen materials.

Section 4.5 contains documentation of the brace construction phase of the project and the installation of the brace into the supporting frame. This section contains the methods used for the application of de-bonding agents and the pouring of the concrete.

4.2 Detailing Procedure

The design forces used to design the connections of the sub-assemblage specimens are obtained from the high-rise design study case presented in Chapter 3. The configuration design chosen and detailed in this chapter are not necessarily the most efficient or designed especially for the BRB system, but have been used extensively in New Zealand and are simple to follow.

The first section of the detailing procedure deals principally with the restraining requirements by the BRB, including clearance between the core braces and encasing members. It also
shows consideration required for the projected part of the BRB element. The main focus of the second section of the detailing procedure is the design of the pinned and bolted gusset plate connections. The design procedure for the connections between beams and columns is also presented.

4.2.1 BRB Detailing

Yielding Segment
The steel core is the restrained yielding segment of the BRB system. As described in the literature review, there are many shapes of the steel core, with the most common being cruciform, rectangular, and circular. The ease of construction has been the major factor in deciding which cross-section is used in this design procedure. The circular steel rod requires more difficult connection details to connect the restrained yielding segment to the restrained non-yielding segment, and is therefore not preferred. Initially, the cruciform shape was chosen as the cross-section area to provide a homogeneous cross-section throughout the BRB elements and to give an equal radius of gyration about both principal axes; however it was realised in the construction phase that, for this to happen, continuous welds would be needed between the three plates of the cruciform core, which introduces the potential for stress concentration-induced defects. Also, it was difficult to get a sufficiently small area of steel with a cruciform section when the thickness of the main element must be at least as thick as the gusset plate. Given that requirement for setting the thickness of the yielding segment, the width was then determined from the brace seismic design action and generated a rectangular cross-section in accordance with clause 12.5.2.2 of NZS3404 (1997) regarding section geometry requirements for yielding regions.

Figure 4.1 shows the restrained yielding segment (steel core) of the BRB system and should be used as a reference for the other elements referred in this section.
The thickness of the rectangular steel core is related to the thickness of the gusset plate. Due to the method by which the unrestrained yielding segment will be connected to the gusset plate thickness (Figure 4.6 and 4.9), the thickness of steel core which is continuous through the brace must be equal to or greater than that of the gusset plate. Therefore, the design of gusset plates are likely to be performed first before the final cross-sectional area is chosen. The guide on choosing the gusset plate thickness will be included in Section 4.2.2. The common available thicknesses of steel plate are 10, 12, and 16 mm. For the sub-assemblage specimen, a thickness of 10 mm has been used for the yielding segment. It is possible to have a thicker yielding core than the gusset plate through the use of packer plates, which must be symmetrical on each side, but it is not desirable to have a thinner yielding segment compared to the gusset plate.

The orientation of the core can be varied (SEAONC, 2001), and the sub-assemblage specimens designed for this project have used the traditional orientation of the concentrically braced frame (CBF), where the flat plate is perpendicular to the floor.

The length of the yielding steel core will be dependent on the total brace length, the required length of the transition regions (the non-yielding regions within the restraining steel tube) and the length of the connection region. There is no required minimum ratio between the thickness and length of the steel core, unlike the cruciform area, where all rectangular plates should be in approximately a 1: 4 ratio.

The length of BRB steel core may affect the energy dissipation capability of the system. A literature review shows that no specific research has been performed or design criteria set to
determine the length of the steel core. Mirtaheri et al. (2010) found that the length cannot be
determined by the energy dissipation efficiency factor alone, due to the risk posed to the
overall structural stability of the system. He recommended the proposed length of 1 m for a
typical building with a 3 m height and 5 m span, but further research and testing are required
to find the optimised configuration. Another method that could be used is to calculate the
design inelastic extension of the yielding core, $\Delta l$, when the design inelastic inter-storey drift
is developed, and to ensure that the ratio of $\Delta l$ to the length of the yielding core is limited to a
small percentage, say, less than 1.0%. This is easily achieved with a yielding core length of
more than 2 metres.

In these proposed detailing requirements, the length of the steel core will be the maximum
allowed after the other lengths of the BRB element are determined. Therefore, the length of
the steel core is the length required to connect the restrained yielding segments from one side
to the other after all other structural member sizes are determined and clearances requirements
are satisfied.

For the two braces tested in this project, the steel core has a width of 70 mm and thickness of
10 mm. The length of yielding segment of the bolted specimen is 3.5 m, while the length of
the pinned specimen is 3.914 m. For these braces, the largest inelastic displacement is 6.7
mm, and therefore the ratio of $\Delta l$/yielding length is 0.19% and 0.17% for bolted and pinned
specimens, respectively.

**Non-yielding segment**
The non-yielding segments, commonly referred to as the brace projection of the BRB
elements, can be divided into the restrained and non-restrained sections. The main purposes of
the non-yielding elements are:

1. To restrain the plastic deformation to the yielding segment of the BRB. For this, they have
to be designed for the overstrength capacity of the yielding element and detailed so as to
ensure a smooth flow of forces from the yielding element into the non-yielding element,
and to prevent local buckling of the projection of the yielding element beyond the
restraining element;
2. To transfer any bending moment in the plane of the frame arising from rotation of the supporting gusset plates into the restraining steel tube, so that there is no bending moment transferred into the yielding core. This is achieved by having a close fit between the sides of the non-yielding segment and the walls of the restraining steel tube, plus a continuous surround of the grout in this region. It also imposes a minimum length requirement on the non-yielding segment, which is given on the next page.

The cross-section of the restrained non-yielding segment is a cruciform, which complies with clause 12.12.7.2(d) of NZS3404 (1997). The vertical part of the cruciform cross-section is a projection of the steel core as shown in Figure 4.2. The side reinforcement, referred to as the wings, will then be welded parallel to the brace along the centre of the vertical part of the non-yielding segment, creating the cruciform area. These wings must have a smooth transition into the yielding core at the ends adjacent to the yielding core, and there must be a gap allowed between this end and the mortar to accommodate compression movement of the core element.

![Figure 4.2: Main plate of the bolted specimen](image)

Unlike the traditional BRB design shown in Figure 4.1, where the non-yielding segments’ width is increased outside the steel tubes or restraining members, the proposed design of the sub-assemblage specimens increases the width of non-yielding segments inside the encasing mortar. The restrained non-yielding segment width will have a gap of 1.5 mm from the steel tube on each side. This is done to improve the stability of the system by preventing twisting of the steel core. The steel core can only move slightly before it will be stopped by the bearing strength of the restrained non-yielding segment plate to the steel tube, due to the small gap between the two elements. This configuration provides resistance to the in-plane rotation of
the bolted connection caused by the frame’s lateral movement of the bolted connection which will be explained later in the chapter. The width of the non-yielding segments is thus limited to the size of the steel tube which restrains them.

The wings of the projected part of the BRB elements are also used to resist the out-of-plane buckling of the non-yielding segments. There is no check made in this design procedure on how much reinforcement is required to prevent the out-of-plane buckling, and therefore the length of the wings is the same as the length of the non-yielding segment. The thickness of the wings is taken as the next thickness below that of the main plate; in the two specimens tested, the thickness of the main plate is 10 mm, and therefore the wings’ thickness is 8 mm.

The non-yielding segments’ length is important, as the element must have a sufficient length to transfer the end moments into the restraining steel tube, without the cruciform end jamming in the tube. The exact length necessary for this has not been determined; this design procedure used a length equal to 1.5 times of the dimension of the steel tube in the plane of the frame, and has performed well. This is therefore the minimum non-yielding segment length recommended within the restraining tube.

The slope through the transition zone from the steel core to the non-yielding segment must not exceed the required increase-in-width to the horizontal length ratio of 1 to 2.5 in accordance to clause 12.12.7.2(e) of NZS3404 (1997). This is important as notched regions are subjected to very high inelastic strain and concentrated yielding, which can lead to brittle fracture and local buckling in compression, respectively. For the sub-assemblage specimen, the notched region transition zone has a ratio of 1:4 on vertical: horizontal. In addition, the beginning and end of this sloping region must be smoothly rounded for a radius of 10 m to fulfil the above clause. This was used in both specimens tested.

As observed in Figure 4.2, the size of the bolted specimen’s non-yielding segment can be increased due to detailing requirements of the bolt. The detailing of the bolt can be found in the steel standard NZS3404 (1997) in accordance to section 9.6, and also governs the size of splice plates used to connect the non-yielding segment to the gusset plate. It is found that the width of the non-yielding segments does not provide the necessary space for the splices to be connected, and does not fulfil the edge distance requirement. One solution to this problem is
to increase the size of the steel tube, which thus increases the corresponding size of the non-yielding segment. In this proposed design procedure, however, the width of the non-yielding segment is further increased to fulfill the bolt detailing requirements. The length of the projected non-yielding segment will be determined by the required distance between bolts and by the increase-in-width to length ratio, which should not exceed 1 to 2.5, in accordance with clause 12.12.7.2(e) of NZS 3404 (1997).

The bolting requirement does not need to be considered for the pinned specimen, and therefore it has not been designed with a projection of its non-yielding segments. This has the effect of making the yielding length and the length of steel tube of the pinned specimen to be longer than the bolted specimen designed using the proposed procedure. However, it is critical that the wings of the cruciform section extend up within 10 mm of the edge of the pin hole to avoid the main plates buckling out of their plane when loaded in compression.

**Steel Tube**

The steel tube acts as a restraint to prevent the overall flexural buckling of the BRB. Its length must encompass the steel core and the restrained non-yielding section of the BRB elements.

In addition to the stiffness requirement check to determine the sizes, studies have found that the thickness of the steel tube contributes to the performance of the BRB system. Takeuchi et al. (2010) showed that in uniaxial cyclic loading tests, specimens containing a rectangular tube with a width-to-thickness ratio of 65:1 experienced local buckling failure. However, specimens composed of a rectangular tube with a width-to-thickness ratio of 25:1 did not show evidence of local buckling failure. It was also found that BRB specimens with 4 mm- and 5 mm-thick steel tubes achieved a 34% and 54% higher compressive strength, respectively, compared to that restrained by a 3 mm thick tube (Young, 2007).

For the sub-assemblage specimen, the size of the tube is increased from the initial $100 \times 100 \times 3.0$ SHS to $125 \times 125 \times 4.0$ SHS to decrease the changes of width that have to be made from the non-yielding segment to its projection to fulfill the bolting requirements. The pinned specimen did not encounter this problem; however, for the sake of comparison, the bigger steel tube is also used. The steel tube used has a thickness of 4 mm and has a width-to-
thickness ratio of 31.25:1. It is therefore recommended that the limits from NZS3404 Table 6.2 for a flat plate element with both longitudinal edges supported are used.

**Cross-Section Summary**

Figure 4.3 shows the cross-section shapes and sizes of the unrestrained non-yielding segments, followed by the restrained non-yielding segments, followed by the yielding segment itself. The projected non-yielding segment has been excluded in the figure.

![Figure 4.3: Summary of cross-section output](image)

**4.2.2 Connection Detailing**

This section covers all the connection types needed to be designed for the two sub-assemblage specimens. The first specimen will be connected to the surrounding frame by a pinned connection, and the second with bolted connections. The welded connections have not been considered due to limited funds needed to construct additional BRB specimens. The design of the gusset plate will also be discussed. The detailing and design of these connections for specifically the BRB system is important as there is scarce information regarding this topic.

A moment endplate has been used to connect the beam and column, while a baseplate has been used to connect the column to the strong floor. The design and detailing of these two types of connections are well known and they will be only briefly explained in this section.

Proper detailing in all these connections is essential to maintain their integrity when the structural members of the system undergo inelastic action and deformation. The design and detailing recommendations herein are intended as supplementary to the connection design requirements published by HERA and SCNZ, especially HERA Report R4-142 (2009) and SCNZ Steel Connect (2007).
**Pinned Connection**

The pin will be designed in accordance to clause 9.5 of NZS3404 (1997). The pin is subjected to shear, bearing, and bending forces which originate from the overstrength design force of the brace. The horizontal component of this overstrength force will be taken as the design shear force, while the vertical component is the design bearing force; however, the standard does not specify on how to determine the design bending moment.

HERA DCB no.32 (1997) provides guidance to determine the design bending force for the pin. Assuming that the pin in bearing resists a uniformly distributed load from the brace force, these forces are transmitted out of the pin in the form of the distributed load shown in Figure 4.4(a). The maximum moment can be found using Equation 4.1 and Figure 4.4 (b) should be used as a reference. The pin allowing for rotation should be designed with a $k_p$ of 0.5 for the nominal bearing capacity (NZS 3404, 1997). This is very important to prevent minor ovalling developing under inelastic cyclic loading; the pins in the experimentally tested braces were not designed for this, developing some ovalling and subsequent slackness during the testing.

$$M = \frac{F}{8} (2a + b + 4c) \quad (4.1)$$

![Figure 4.4: (a) Load distribution of a pin in bearing and (b) configuration of a generic pinned specimen](image-url)
The clamping plate shown in Figure 4.5 has been designed to provide a load path from the non-yielding segment to the pin. The size of the designed pin is Ø70 mm and a hole with 0.5 mm clearance has been made. It is important to specify the clearance as the tolerances on the hole diameter must be more precise than normal structural work clearance. The gap located at the front of the plate is made to provide access for the non-yielding segment wings to be welded to the clamping plate (for location of the weld see Figure 4.6 and 4.7). The gap length of 155 mm is provided to allow a 150 mm fillet weld on each side and a 5 mm specified gap between the gusset plate and the yielding segment. The welding between the wings and the non-yielding segment has to be flushed, as there is no space available between the clamping plate and the wings for any residue of fillet weld. Any extension of the wings are fillet welded to the connecting plates. Bear in mind that Figure 4.6 shows the final construction drawing after the wings have been extended to within 10 mm of the edge of the hole to prevent the buckling of the clamping plate found in the experimental tests.

Figure 4.5: Clamping plate dimension

A pin connection in a tension member has to be detailed in accordance to clause 7.5 of NZS3404 (1997). The clamping plate has been designed to comply with this requirement. Firstly, the thickness of the clamping plate is 8 mm (Figure 4.6) which is greater than 0.25 times the 30 mm edge distance. The dimension of the clamping plate has also been specified to fulfil the requirement stated in 7.5 (d) regarding the sum of areas of material on each side of the hole for a pin. In this case the net area required for the member is the area of the steel core.
Chapter 4: Detailing and Construction of the BRB Test Specimens

Bolted Connection
The bolted connection is easier to detail and design in comparison to the pinned connection. Clause 9.3.2.2 of NZS3404 (1997) regarding the designing of bolts in tension has been followed in this proposed design procedure. To be conservative, the wings are assumed not to take part in force transfer between the non-yielding segment and the gusset plate, but should be considered when designing for slip prevention as the wings do share the load, even though
the amount of load they resist cannot be determined easily. Therefore, only the bolts located in the same plane as the steel core are considered to resist the design force in this proposed design procedure.

When choosing the bolt size, the clearances required to install and fully tension the bolts will determine the outstand dimensions of the steel plates. This was observed too late in the experimental test when the plate sizes were slightly too small to allow the M16 8.8/TB bolts to be fully tensioned. These requirements are stated in section 9.6 of NZS3404 (1997). The size of the splice may also, in turn, increase the size of the gusset plate. A bolt diameter of 16 mm was chosen for the specimen design as it is the minimum recommended bolt size for structural use, and allowed the minimum gusset plate and bolted connection steel element sizes to be used. However, the spacings were such that the bolts could not be fully tensioned, and in two rows, not even installed with nuts, but had to be installed into threaded holes in the plates.

In the design of the bolted connection, a check should be made in the force generated by the overturning moment of the bolted connection. This moment is caused by the lateral forces generated by the restraining members to prevent the steel core from buckling. The centre of rotation is located in the middle of the bolted connection and the forces generated are resisted by the bearing strength of the restrained non-yielding segment and the gusset plate, as shown in Figure 4.8 This check estimates the lateral force as 2.5% of the overstrength brace design force. It is quite hard to accurately estimate the effect of this overturning moment; however, it is not likely critical with proper detailing of the BRB element.

![Figure 4.8: Deflected shape](image-url)
The configuration of the bolted connection can be seen in Figure 4.9.

---

**Figure 4.9: Bolted connection details**

**Figure 4.10: Welding between gusset plates**
**Gusset Plate Connection**

The design example contained in section 61.1 of DCB no. 61 (2001) has been followed to design the gusset plate. The angle of the brace in the sub-assemblage specimen is $41.67^\circ$. In this case, the BRB specimens have not been repositioned to make the connection more compact and cost-effective, and the centreline of the brace meets with the centreline of the gusset plate. Figure 4.9 shows that the bolted connection has a positive noding eccentricity, but the forces have been proportioned so the design can proceed with the assumption that the centrelines of brace/beam/column intersect at the work point (WP).

The check performed for the endplate tension capacity requires that the mode 3 failure of no flange yielding and full bolt extension is prevented. The mode 3 failure, however, is rarely critical; the critical modes of failure are usually mode 1, with complete flange yielding and no bolt extension, or mode 2, with partial flange yielding and partial bolt extension. For the sub-assemblage specimen, the endplates connected to the column are governed by mode 2 failure, while the endplates connected to the beam are governed by mode 1 failure. This means that the connection to the column will allow partial bolt extension under severe loading. In order to suppress opening up along the bolt line, it is recommended that the bolt size in all the semi-rigid connections be such that mode 1 governs all out of plane connection plate designs.

As seen in the detail of Figure 4.10, the side and bottom/top endplates are connected by fillet welds. The welding details will allow the gusset plates to remain flexible under severe earthquake loading by opening when the brace is compressed and closing when the brace is stretched.

In addition to the design requirement, the gusset plate size and thickness is governed by the size of the connecting/clamping plate, detailing requirements for pin and bolt, and the yielding segment thickness if it has been decided first. As a rule of thumb, gusset plates should have sufficient thickness to prevent large distributed stresses, but stiff enough to prevent large stress concentrations that may cause out-of-plane buckling. It is also recommended to make the gusset plate as compact and efficient as possible, as stronger gusset plates requires thicker welds and have less stiffness and ductility required for a seismic resisting system. The designer can also decide to create various shapes of gusset plates to move the centrelines of the BRB or to provide space required to install the connections.
The gusset plate details of the pinned connection are shown in Figure 4.6 and the bolted connection is shown in Figure 4.9. The gusset plate is connected by fillet welds on each side by the endplate and has an $f_y$ of 260 MPa. The gusset plate is rectangular with its top corner cut at a $41.67^\circ$ angle to the non-yielding segment.

The hole on the gusset plate of the pinned connection is located at the centre of the gusset plate, and has to be cut with a 0.5 mm clearance rather than the nominal hole clearance specified in NSZ3404. Compared to the pinned connection, the gusset plate for the bolted connection is harder to detail, as each bolt hole must be carefully specified to ensure that they will be able to line up with the holes on the non-yielding segment. In addition, the welding on the gusset plate wings shown in Figure 4.11(b) has to be performed with utmost care, and a trial erection may be necessary to ascertain where the winged plates should be welded. The weld will be a fillet weld which does not have to be flushed as there is a gap between the spliced or connecting plates.

![Figure 4.11: (a) Pinned connection gusset plate and (b) bolted connection gusset plate](image)

**Moment Endplate**

The design example of a brace/beam/column connection contained in section 5 of DCB no.61 (2001) outlines the method used to design the moment endplate connection and is followed in the design of the sub-assemblage specimen. For the design of the BRB system, the check
made for the adequacy of the endplate in tension is important. The moment endplates used in this design are flushed to provide space for the gusset plate connection. This type of connection is chosen as it has the ability to function as a semi-rigid connection under higher levels of seismic loading, while still providing a reasonable stiffness.

When the brace undergoes compression, the moment endplate connection will be subjected to a combination of tension and shear forces. In the design of the moment endplate for the sub-assemblage specimen, the check (Equation 4.2) performed to account for these combined forces found that the final moment endplates design is just slightly under the required capacity for this check.

\[
\left(\frac{V_{\text{total brace,com}}^*}{\phi V_{\text{fn}}}\right)^2 + \left(\frac{H_{\text{beam brace,com}}^*}{\phi N_{\text{tf}} \times 1.4}\right)^2 \leq 1.0
\] (4.2)

Another important check, which is not included in the DCB No. 61 (HERA, 2001), is to determine the column flange tension adequacy. This check is performed using the same method outlined in section 4.2 of DCB No. 61 (HERA, 2001). Similar to the gusset plate, it is recommended that mode failure no.3 is suppressed. In addition, the designer should know which mode of failure governs the system as it affects the system performance.

The moment endplate details are shown in Figure 4.12. The endplates are connected to the beam web by fillet welds, while the flanges of the weld are butt-welded to the endplate. The bolt size should be such that mode 1 failure governs, thus avoiding any extension at the bolt line under axial or bending induced tension.

**Baseplate**

The baseplates are designed specifically for the sub-assemblage specimen and its design should not be considered the same for other applications. The design of the baseplate uses the procedure concept found in HERA DCB No. 56 (2000). As mentioned in the design bulletin, the baseplate must be able to resist the capacity design derived action and internal action due to column inelastic demand. As the specimens are created to fully measure the BRB performance in under any way that the baseplate of the column will fail, when the frame is tested, the failure mode should be suppressed if possible. Horizontal sliding between the baseplate and steel plate should also be avoided as it will affect the performance of the whole seismic resisting system.
The sub-assemblage specimen column-to-baseplate connection has been designed in two different baseplate connections. The column to baseplate connection beside the bottom gusset plate connection has been connected by bolts, while the other columns are connected to the baseplate via welding. The combination of both these connections gives both columns a pinned restraint which is intended for the experimental testing.

The bolted baseplate details are shown in Figure 4.6 and 4.9, while the welded baseplates details can be seen in Figure 4.13.

**Stiffener**

Eight stiffeners have been designed for the sub-assemblage specimen. These have been designed to prevent the yielding of column flanges from the actuator force.
4.3 Clearance Requirements

In detailing the BRB system, allowance for absorbing the steel brace inelastic deformation is very important. In Figures 4.6 and 4.9 a 5 mm connection gap, 15 mm gap bearing and 20 mm contraction allowance has been specified in the assembly drawing. This section explains the general clearance requirements which have been considered in the detailing process.

4.3.1 Connection Gap

The system will experience inelastic deformation under severe earthquake loading; therefore, a gap must be provided to prevent collision between the BRB elements and the gusset plate. This is undesirable, as the BRB elements will directly bear against the gusset plate, which may cause the gusset plate to buckle. This gap requirement is generally small, as it only accounts for the slack that may happen in connection and rare cases where the connecting/clamping plates buckle slightly under loading.
4.3.2 Gap Bearing

A gap shown in Figure 4.14 is also required between the yielding segment and non-yielding segment of the steel core to prevent load transfer from the steel core to the mortar. Fussell suggests the use of Equation 4.3 to find the required gap bearing. The step-by-step method can be found in Fussell’s (2010b) paper on the design of buckling restrained brace systems.

\[ \Delta_{b,\text{max}} = 1.25 \left( L_{b,\text{max}} - L \right) \]  \hspace{1cm} (4.3)

The \( \Delta_{uls} \), in this case, has been obtained through the aid of the SAP2000 program. A subassembly model has been constructed with the appropriate base restraint. The earthquake force subjected to the frame is the overstrength design brace, which is approximately 250 kN and 180 kN in the lateral horizontal direction (Figure 4.15). The elastic deflection is found to be approximately 10 mm laterally and 8 mm in the brace direction, and thus the gap bearing should be more than 10 mm. In this case, the gap bearing has been specified as 15 mm to account for inelastic behaviour.
4.3.3 Contraction Allowance
The contraction allowance provides sufficient clearance width to avoid collision of the brace elements with surrounding frames. Xie (2005) recommends that the allowances should be decided according the inter-storey drifts of the structure under earthquake effects. The largest inter-storey drift of the structure is found to be 8.8 mm. These values are likely to be larger for a single storey frame and further consideration is needed for beam and column deflection; therefore, a 20 mm contraction allowance was specified for the tested components.

4.4 Construction Material Specification
This section provides recommendations and guidance on material selection. It also presents general material data for the specimens tested, including the steel properties, de-bonding agent and filler material.

4.4.1 Steel
The steel has been provided and fabricated by Grayson Engineering Ltd. Grade 300 steel was chosen for the design of these braces. This has a nominal yield strength of 300 MPa and was used to design the components; however, the strength of the actual material used will be different and has been determined by tensile testing.

The tensile test was performed with the assistance of the Department of Chemical and Materials Engineering, Faculty of Engineering, University of Auckland. The tests were conducted by using the machine shown in Figure 4.17(a). Three samples of the steel plates used to construct the specimens’ steel core have been obtained and cut according to the dimensions shown in Figure 4.16. The first sample was tested at a rate of 0.01 mm/s and the rest of the samples were tested at a rate of 0.055 mm/s until failure (Figure 4.17 (b)).

The tensile test results conclude that approximately 44 kN is required to break approximately 125 mm$^2$ of cross-sectional area, shown in Figure 4.18. The average yielding strength of the steel used to construct the specimens determined from these tests is 350 MPa. For the calculation procedure and the measurement of each specimen see Appendix D.
Figure 4.16: Tensile test specimen dimensions

(a) Tensile test machine and (b) broken specimens samples compared to the initial condition

Figure 4.17: (a) Tensile test machine and (b) broken specimens samples compared to the initial condition

Figure 4.18: Tensile test results of displacement vs. axial load
4.4.2 De-bonding agent

A de-bonding agent is required between the core braces and the restraining elements of the BRB. This material will allow the brace to slide freely inside the encasing grout when it compresses or stretches under loading. Selection of a suitable material is the key to the successful performance of the BRB. In this research project, it was decided to use petrolatum tapes. Denso Ltd, a company which specialises in corrosion prevention and sealing technology, was consulted to choose a suitable petrolatum tapes type. The following criteria have been adhered to in the choosing of the appropriate petrolatum tape to be used in the experiment:

1. The tape must be able to be easily wound around the steel plates with the wings attached and cover the entire central yielding flat strip up to the beginning of the cruciform sections at the ends;
2. It must form a soft layer between 2 mm to 3 mm thick between the steel and grout surfaces to allow for the Poisson Ratio induced increase in thickness when the steel core expands under inelastic compression, but it cannot be too thick such that it allows the yielding segment to buckle slightly before restrained by the grout;
3. It must completely de-bond the steel from the concrete and do this without being compromised when the grout is placed around it. For example, the grout must not remove the impregnated petroleum product in the tape.
4. The petroleum product must not be leached into the ground and compromise its properties;
5. The tape must dependably remain in place when the grout is poured.

Denso Ltd has recommended the usage of their product Denso tape. The full product data sheet is contained in Appendix D. The tape can also act as corrosion prevention if the BRB frame is placed outside. The material performed fully to expectations and is recommended as the de-bonding agent.
4.4.3 Filler Material

In the construction of the sub-assemblage specimen, normal concrete mix or grout has been chosen as the appropriate filler material. Sika NZ, a company known for its specialities in ready mix concrete, has been consulted on which of their products will be the most suitable for the two BRB specimens. The following criteria have been adhered to in choosing the most appropriate grout or concrete mix to be used in the experiment:

1. The filler material’s main purpose in the BRB system is to prevent buckling of the steel core. 25 – 30 MPa is considered sufficient to prevent the steel core from global and local buckling (Gheidi et al., 2009);

2. The material must be able to self-compact as there will not be enough space to use concrete vibrators to aid the compaction;

3. To ensure the workability of the material, a 60 mm slump is recommended as the filler must readily form around the steel core without segregation of excessive bleeding and be easily pourable;

4. It is very important that the grout has the ability to counteract natural shrinkage. If, shrinkage happens, the concrete will not fully encase the steel segment and may not perform as expected. On the other hand, a small expansion of concrete can be allowed, but should be kept to a minimum, and this expansion will then be counteracted by adding an extra layer of the de-bonding material.

5. There are only small spaces between the steel tube and the steel core in both the yielding and non-yielding segments. In addition to that, the concrete must also be able to fully encase the steel core wrapped in the de-bonding material. Therefore, the grout mix must contain very fine natural aggregates.

Sika NZ has recommended the usage of their product Sika Grout 212, which is a high strength, shrinkage-compensated, pourable cementitious grout. The full product data sheet is contained in Appendix D.
4.5 Construction Phase

As mentioned earlier in this chapter, two BRB specimens were constructed for the experimental test. In addition to the two test specimens, a frame was constructed to act as the experimental test rig and to ensure that the support conditions at the ends of the brace, and therefore the actions generated on the brace, were realistic. This section contains the specific construction phase of the BRB system which requires special attention. All construction, apart from the grout pouring, was performed by personnel at Grayson Engineering Ltd.

4.5.1 Trial Erection

It is recommended that a trial erection is performed on any BRB and this was done on both specimens (Figure 4.19). The pinned specimen, due to its nature, only has a small tolerance and the trial was critical to ensure that the pin could fit between the clamping plate and the gusset plate. The bolted specimen needed the trial erection, principally to determine the location of the gusset plate wings but also to check that the bolt holes were correctly positioned.

![Figure 4.19: Partially erected, bolted and pinned specimen](image)

4.5.2 Gap Bearing

The gap bearings are formed using the polystyrene material and put after the primer is applied to the surface of the steel, but before the steel plates are wrapped with the Denso tape (Figure 4.20). Wrapping the polystyrene with Denso tape will ensure that the gap bearing will be located at the correct place after the grout is set. Polystyrene as a gap bearing material is chosen as it is easily shaped, and Figure 4.14 shows the placement of the gap bearing. Note
that, even though the polystyrene located at the slope is not rectangular, it still must have a thickness of 15 mm in the direction of the steel core.

4.5.3 Denso Tape Wrapping
Before the Denso tape is wrapped, it is recommended to prepare the steel surface by brush wire cleaning, followed with the application of a thin film of DENSO MP Primer. The Denso tape is able to be wrapped horizontally and should not stretch, with at least 55% overlap between two sheets of the Denso tape. The tape will then be smoothed over by hand, or using a roller to ensure a homogeneous surface. Refer to Figure 4.21 and 4.22 for this construction phase. The aim with these test specimens was to create a uniform layer of Denso tape between the steel core and the concrete grout that varied between 1 and 2 mm in thickness.

Calculation of the de-bonding thickness can be done by initially assuming that the most severe lateral displacement or drift experienced by the sub-assemblage frame is 2% of the storey height. From this, with the assumption of a rigid body, the decrease in length of brace can be found and, assuming that only the yielding segment has deformed under loading, the cross-sectional area of the steel core after loading can be found using Equation 4.4. The changes in the width and thickness of the steel core can then be approximated using the Poisson Ratio. It is also noted that the pinned connection specimen will need to be wrapped with less Denso tape as its changes of cross-sectional area is less compared to the bolted specimen due to the difference between the yielding segment length of both specimens. In addition, the thickness of the Denso tape must also be increased to account for the expansion
of the grout. This method of estimation is very rough and the results should not be taken as an exact value.

\[
\frac{A_i}{L_i} = \frac{A_d}{L_d}
\]  

(4.4)

where

- \(A_i\): Initial cross section area of the brace
- \(A_d\): After deformation cross section area of the brace
- \(L_i\): Initial cross section area of the brace
- \(L_d\): After deformation cross section area of the brace

Figure 4.21: Overlap sheets of Denso tape (left) and final look after (right)

Figure 4.22: Before (left) and after the Denso tape wrapping (right)
4.5.4 Grout Pouring

The grout pouring was conducted by the grout provider Sika NZ in the Grayson Engineering workshop. Pouring was performed by sealing or covering one end of the tube and welding the other side in a rig, which will enable the grout to be poured from an elevated position by using buckets. The grout was mixed by adding a necessary amount of water to make it workable using a low speed electrical drill. Gravity will ensure an even distribution of the grout throughout the inner tube. Refer to Figure 4.23 to 4.27 for the grout pouring process.

It is important that the steel plates must be in tension and centred as closely as possible within the confining steel tube when concrete is poured, as this will ensure that the yielding segment will be straight during grout placement and setting. Any out of straightness of the yielding core will magnify the tendency for instability and compressive buckling of the brace when loaded in an inelastic manner. There is also concern of trapped air, which cannot be allowed in the mortar filling; this can either be treated by providing an air entraining agent in the grout mix, or by pumping the concrete from the underside of tube.

For these two specimens, the construction took approximately one hour each with two people for two specimens, and was physically demanding as the bucket has to be hauled to the pouring position. Therefore, for multiple construction of BRB specimens, it is recommended for pumps to be used rather than buckets.
Chapter 4: Detailing and Construction of the BRB Test Specimens

Figure 4.24: Sika Grout 212 Package (left) and grout mix (right)

a) Temporary frame
b) Top support
c) Bottom part

Figure 4.25: Grout pouring rig

Figure 4.26: Pouring grout using bucket (left) and fully grouted specimens (right)
Chapter 4: Detailing and Construction of the BRB Test Specimens

The specimens must then be left for one to two days for the grout to achieve a strength of 20 MPa. Therefore, the amount of time that the specimens are hanged on the rig depends on the grout ability to achieve strength, and considering efficiency it is recommended, to use a grout or concrete which can achieve a 20 MPa compressive strength in 24 hours.

Figure 4.27 shows the specimens after transportation to the University of Auckland test hall. The steel plates are not cast exactly in the centre and are slightly twisted; however, this did not cause any problems as there was a solid layer of concrete between the inside face of the steel tube and the Denso tape-covered surface of the steel plates. This angle of twist was less than 5 degrees but was visible, especially in the pinned brace. Note that the pinned brace connection shown in Figure 4.27 did not have the side plates continuous up to the face of the pin. As described in Section 5.4.3, this generated a buckling failure of the pinned plates on the first cycle of compression loading, requiring side braces to be retrofitted before the test continued. The retrofitted detail performed very well with no buckling.

![Figure 4.27: Results after 3 days](image)
Chapter 5
SUB-ASSEMBLAGE TESTING

This chapter investigates the behaviour and performance of the BRB sub-assemblage specimens through experimental testing. These tests are fundamental to confirming the reliability of the proposed design and detailing procedure outlined in the previous chapter. The testing was carried out in the Civil Engineering Test Hall at the University of Auckland. The test setup and loading regime used for the experimental tests are presented in this chapter, followed by the results and discussion.

5.1 Introduction
The sub-assemblage specimens are designed to resist lateral forces induced by earthquakes and to behave and perform like a generic BRB system. In the experimental test, the earthquake forces were simulated using a Shore Western Short Ended 923.5-22.58 dynamic actuator. Only 2 specimens with different connection types have been constructed for the experimental test due to the cost and time required for construction, and hence these two specimens were used for the entire experimental test with no replacement. This was less than ideal as each subsequent test will bear some of the effects of the earlier tests; however it was also a good opportunity to examine the ability of the BRBs to withstand multiple severe events separated in time, which will be required if the braces are to remain in place, following a severe earthquake.

Each specimen was subjected to three different loading regimes: trial static, cyclic static (at pseudo-static rates of loading, being 1/50\textsuperscript{th} of the dynamic rate) and cyclic dynamic (at earthquake rates of loading, typically 1 second per cycle in the inelastic range). The magnitudes of the loading regimes are based on the document released by SEAONC (2011) on the testing of BRB systems. The specified loading was applied by the dynamic actuator shown in Figure 5.1, which has a maximum capacity of 250 kN (rated at 300 kN, but the pump size is too small to allow the rated capacity to be developed), and is controlled by displacement. The test series was performed in three weeks’ time, with the exception of the
strain ageing test. In chronological order, the first test conducted was the trial test of the pinned specimen, followed by both bare frame tests. A second static trial test was performed on the pinned specimen, and the cyclic static test of the pinned specimen followed after. The bolted specimen was then installed and tested for all of the loading regimes. Lastly, the pinned specimen was dynamically tested and left for two and a half weeks in an average and near constant temperature of 22 °C, before it was tested with the same dynamic loading regime to investigate the effect of strain ageing.

5.2 Installation and Test Setup

The installation was performed by two people, with the use of a crane. The gusset plate was connected to the BRB specimen before the specimen was lifted into place using the crane. The gusset endplate was then bolted to the column and beam before the crane was released.

5.2.1 Overall Test Set Up

![Elevation plan of the test setup](image-url)

Figure 5.1: Elevation plan of the test set up
Figures 5.1 and 5.2 show the elevation and floor plan of the test setup. The test rig was supported on a customised baseplate, with holes which lines up with stress bar holes on the strong floor. Unless otherwise stated, the column positioned near the actuator will be referred to as the left column and the column positioned further will be referred to as the right column.

5.2.2 Restraining Mechanism
In the initial set up, two restraining mechanisms were used. The first restraining mechanism was the stress bar used to restrain the baseplates from moving when the experimental test commenced (Figure 5.3). The second restraining mechanism was a steel rod, which acts as a lateral restraint to keep the rig from twisting when the horizontal force was applied to the top of the left column (Figure 5.3).
5.2.3 Instrumentation

A series of instruments were used to monitor and record the performance of the sub-assemblage specimen. Two Linear Variable Differential Transformers (LVDT) were used to measure the lateral deflection of the sub-assemblage frame. As shown in Figure 5.4, one is built in the actuator and the other was installed at the top of the right column. The actuator has a built-in load cell to measure the forces applied to the frame. Two turn pots were used to measure the in-plane and lateral buckling at the centre of the specimen, and portal gauges were used to determine whether the baseplates moved during the experimental test (Figure 5.5). Initially, strain gauges were planned to be installed to measure the strain throughout the specimen and connections; however, due to administration error resulting in a delay and the incorrect number of strain gauges delivered, no strain gauges were used in these experimental tests. All measurements were recorded by the two computers shown in Figure 5.6.

Figure 5.4: Actuator (left) and LVDT on the top back column (right)

Figure 5.5: Portal gauge connected to the right column (left) and turn pot to measure in-plane movement (right)
5.2.4 Pinned Specimen

There was no difficulty in installing the pinned specimen, as there were no additional preparations needed to be done to the pin, except that it should be evenly divided on each side of the gusset plate, which was done by using a hammer. As the gusset plate of the sub-assemblage frame was designed to be symmetrical, the installation would not be affected if the top gusset plate was installed at the bottom, and vice versa. Figure 5.7 shows the picture of the pinned specimen installed on the test rig. Note that the picture shows the pinned specimen that had been altered due to the circumstances explained in Section 5.4.

Figure 5.6: Actuator controller (left) and data logger processed data (right)

Figure 5.7: Installed pinned specimen
5.2.5 Bolted Specimen

Compared to the pinned specimen, the bolted specimen was more difficult to install, and the bolts were greased before being fully tensioned to prevent torsion failures. However, the initial installation of the bolted specimen also revealed that the current arrangement of the bolted connection did not take into consideration the extra space that is needed between the nuts of the bolts. As a correction, the connecting plates and the wings were threaded to allow the bolts to be installed without their nuts, shown in Figure 5.8 (left).

The bolts of the bolted specimen were unable to be fully tensioned as per requirement of the construction of an earthquake resisting structure. This was also caused by the lack of space between the connecting plates of the wings and primary segment. The bolts, instead, had only been manually tightened with the use of a spanner. This put them only into the snug tight range and is less than required or desired in practice.

To address this problem in practice, it is recommended to take account of this in future detailing of the bolted specimen, to allow extra space between the connecting plates for nuts and application of bolt tensioning tools.

Figure 5.8: Installed bolted specimen