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ABSTRACT

This thesis presents the findings of an experimental and analytical study into the seismic performance of a special steel chevron bracing system. Although very popular as a lateral load resisting system, it is known that concentric chevron braced frames can exhibit poor performance under severe load demands. A vertical slotted connection (VSC) detail was introduced between the top of the braces and floor beam above to prevent vertical load transfer to the beam and limit brace forces to the compressive resistance of the members.

An experimental program was devised to study VSC chevron braced frame behaviour under quasi-static cyclic loading conditions. Full-scale tests were performed on two specimens with hollow tube braces. One of the specimens had the braces filled with concrete. Both frames exhibited stable, predictable behaviour under cyclic loading. The concrete-filled tube specimen sustained higher peak loads, demonstrated greater residual strength and dissipated more energy than the unfilled tube specimen. This was due to the partial inhibition of local buckling by the concrete core. The unfilled braces experienced severe local buckling, which lead to early fracture and eventual failure of the specimen. For both tests, the VSC detail provided free vertical movement of the brace assembly and out-of-plane buckling of the braces did not seem to provide significant restraint.

An analytical study of the VSC chevron braced frame revealed that the flexibility of the connections needed to be included in the computer model, especially for the slotted bolted detail. Models of the frame were developed using two different computer programs, RUAUMOKO and SAP2000, and calibrated to the observed experimental results. The SAP2000 model was able to better represent the backbone curve of the hysteretic behaviour observed in testing. In both models, the use of a horizontal spring element for the brace-to-beam connection was an effective way to model the flexibility in the VSC detail while maintaining a vertical release in the connection. The results of an illustrative example comparing a conventional and VSC chevron braced frame further demonstrated the vulnerability of the gravity frame in a conventional system and highlighted some of the benefits of the VSC system.
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABSTRACT</td>
<td>ii</td>
</tr>
<tr>
<td>TABLE OF CONTENTS</td>
<td>iii</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>vii</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>viii</td>
</tr>
<tr>
<td>ACKNOWLEDGEMENTS</td>
<td>xi</td>
</tr>
<tr>
<td><strong>CHAPTER 1</strong></td>
<td></td>
</tr>
<tr>
<td>INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>1.1 Background</td>
<td>1</td>
</tr>
<tr>
<td>1.2 Seismic Performance of Concentrically Braced Frames (CBFs)</td>
<td>2</td>
</tr>
<tr>
<td>1.2.1 Past Performance of CBFs</td>
<td>4</td>
</tr>
<tr>
<td>1.2.2 Structural Performance of CBFs</td>
<td>14</td>
</tr>
<tr>
<td>1.2.3 Design Requirements for CBFs</td>
<td>16</td>
</tr>
<tr>
<td>1.3 Objectives of Research</td>
<td>16</td>
</tr>
<tr>
<td>1.4 Scope of Research</td>
<td>17</td>
</tr>
<tr>
<td><strong>CHAPTER 2</strong></td>
<td></td>
</tr>
<tr>
<td>LITERATURE REVIEW</td>
<td>19</td>
</tr>
<tr>
<td>2.1 Introduction</td>
<td>19</td>
</tr>
<tr>
<td>2.2 Past Research</td>
<td>19</td>
</tr>
<tr>
<td>2.2.1 Conventional Chevron Braced Frames</td>
<td>19</td>
</tr>
<tr>
<td>2.2.2 Energy Dissipation Elements in CBFs</td>
<td>23</td>
</tr>
<tr>
<td>2.3 Significance of Current Research</td>
<td>29</td>
</tr>
<tr>
<td><strong>CHAPTER 3</strong></td>
<td></td>
</tr>
<tr>
<td>SPECIMEN MATERIAL TESTING</td>
<td>31</td>
</tr>
<tr>
<td>3.1 Introduction</td>
<td>31</td>
</tr>
<tr>
<td>3.2 Tension Tests</td>
<td>31</td>
</tr>
<tr>
<td>3.2.1 Description of Test Set-up</td>
<td>31</td>
</tr>
</tbody>
</table>
5.4.1 Global Behaviour ......................................................... 68
5.4.2 Local Buckling Effects .................................................... 70
5.4.3 Fracture and Failure Characteristics .................................. 72
5.5 Ductility ........................................................................... 73
5.5.1 Strength Degradation ....................................................... 73
5.5.2 Stiffness Degradation ....................................................... 74
5.6 Energy Dissipation Capacity ................................................. 74

CHAPTER 6
ANALYTICAL MODELLING ......................................................... 76
6.1 Introduction ...................................................................... 76
6.2 Detailed Model .................................................................. 76
  6.2.1 RUAUMOKO Computer Program .................................... 77
  6.2.2 Description of Model .................................................... 78
  6.2.3 Brace Hysteresis Properties .......................................... 79
  6.2.4 Analysis Results ........................................................... 80
6.3 Calibration of Model .......................................................... 81
  6.3.1 Comparison to Numerical Prediction ................................ 81
  6.3.2 Parametric Study and Sensitivity Analysis ....................... 82
  6.3.3 Adjustments to Model .................................................. 84
    6.3.3.1 Serial Spring Model .............................................. 84
    6.3.3.2 Reduced Displacement Amplitude Model ................. 87
6.4 Simplified Model .............................................................. 89
  6.4.1 SAP2000 Computer Program ........................................ 89
  6.4.2 Description of Model .................................................... 90
  6.4.3 Plastic Hinge Properties .............................................. 91
  6.4.4 Analysis Results ........................................................... 92

CHAPTER 7
ILLUSTRATIVE EXAMPLE .......................................................... 93
7.1 Introduction ...................................................................... 93
7.2 Description of Building ................................................................. 93
7.3 Description of Model ................................................................. 95
7.4 Analysis Results ................................................................. 96
   7.4.1 Conventional Chevron Braced Frame Model ......................... 96
   7.4.2 VSC Chevron Braced Frame Model ...................................... 98

CHAPTER 8
CONCLUSIONS AND RECOMMENDATIONS .............................................. 101
  8.1 Summary .................................................................................. 101
  8.2 Conclusions ............................................................................. 102
  8.3 Recommendations ................................................................... 105

REFERENCES .................................................................................. 108
APPENDIX A – Instrumentation Details .............................................. 112
APPENDIX B – Fabrication Drawings ................................................. 115
APPENDIX C – Test Photographs ......................................................... 120
APPENDIX D – Numerical Prediction of Test Specimen Behaviour ......... 149
LIST OF TABLES

Table 3.2.1 Material Properties for HSS Tube Wall Test Coupons.......................... 35
Table 3.2.2 Material Properties for HSS Tube Corner Test Coupons ...................... 36
Table 3.2.3 Material Properties for 12.7 mm Plate Test Coupons............................. 36
Table 3.2.4 Material Properties for 19.0 mm Plate Test Coupons.............................. 36
Table 3.3.1 Material Properties for HSS Stub-Columns........................................ 41
Table 4.4.1 Test Instrumentation Set-up.................................................................. 50
Table 4.4.2 Test 1 Loading Protocol......................................................................... 54
Table 4.5.1 Test 2 Loading Protocol......................................................................... 58
Table 5.2.1 Visualeyez™ Displacement Data for Test 2, Step 3................................. 64
Table 5.2.2 Visualeyez™ Displacement Data for Test 2, Step 5................................. 64
Table 5.2.3 Visualeyez™ Displacement Data for Test 2, Step 8................................. 64
Table 5.2.4 Comparison of String Potentiometer and Visualeyez™ System Data ....... 65
Table 6.3.1 Displacement-History for Reduced Displacement Amplitude Model........ 88
Table 7.3.1 Lateral Loading Pattern for Pushover Analysis...................................... 96
# LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1.1</td>
<td>Conventional Chevron Braced Frame Collapse Mechanism</td>
<td>2</td>
</tr>
<tr>
<td>1.2.1</td>
<td>Typical Concentric Bracing Configurations</td>
<td>3</td>
</tr>
<tr>
<td>1.2.2</td>
<td>Conventional Chevron Braced Frame</td>
<td>4</td>
</tr>
<tr>
<td>1.2.3</td>
<td>Foothill Medical Center: Damage after San Fernando Earthquake</td>
<td>6</td>
</tr>
<tr>
<td>1.2.4</td>
<td>Olive View Hospital: Damage after San Fernando Earthquake</td>
<td>7</td>
</tr>
<tr>
<td>1.2.5</td>
<td>Pino Suarez Complex: Column Failure after Mexico City Earthquake</td>
<td>9</td>
</tr>
<tr>
<td>1.2.6</td>
<td>Damage to Braced Frames after Hyogo-ken Nanbu Earthquake</td>
<td>13</td>
</tr>
<tr>
<td>1.3.1</td>
<td>Vertical Slotted Connection Chevron Braced Frame Inelastic Response</td>
<td>17</td>
</tr>
<tr>
<td>2.2.1</td>
<td>TADAS Device in Chevron Braced Frame</td>
<td>27</td>
</tr>
<tr>
<td>3.2.1</td>
<td>Baldwin Universal Testing Machine</td>
<td>32</td>
</tr>
<tr>
<td>3.2.2</td>
<td>Material Testing Coupons for Tension Tests</td>
<td>32</td>
</tr>
<tr>
<td>3.2.3</td>
<td>Custom Grips for HSS Tube Corner Test Coupon</td>
<td>33</td>
</tr>
<tr>
<td>3.2.4</td>
<td>Test Coupon with Attached LVDT</td>
<td>34</td>
</tr>
<tr>
<td>3.2.5</td>
<td>Material Testing Data Acquisition System</td>
<td>35</td>
</tr>
<tr>
<td>3.2.6</td>
<td>Typical Stress-Strain Curve for HSS Tube Wall Test Coupon</td>
<td>36</td>
</tr>
<tr>
<td>3.2.7</td>
<td>Typical Stress-Strain Curve for HSS Tube Corner Test Coupon</td>
<td>37</td>
</tr>
<tr>
<td>3.2.8</td>
<td>Typical Stress-Strain Curve for 12.7 mm Plate Test Coupon</td>
<td>37</td>
</tr>
<tr>
<td>3.2.9</td>
<td>Typical Stress-Strain Curve for 19.0 mm Plate Test Coupon</td>
<td>38</td>
</tr>
<tr>
<td>3.3.1</td>
<td>Stub-Column Test Specimen</td>
<td>39</td>
</tr>
<tr>
<td>3.3.2</td>
<td>Location of Strain Gauges for Alignment and Testing</td>
<td>40</td>
</tr>
<tr>
<td>3.3.3</td>
<td>Instrumented and Whitewashed Specimen in Testing Machine</td>
<td>40</td>
</tr>
<tr>
<td>3.3.4</td>
<td>Typical Stress-Strain Curve for HSS Stub-Column</td>
<td>42</td>
</tr>
<tr>
<td>3.3.5</td>
<td>Failed HSS Stub-Column Specimens</td>
<td>43</td>
</tr>
<tr>
<td>4.2.1</td>
<td>Permanent Testing Frame in Civil Engineering Structures Laboratory</td>
<td>45</td>
</tr>
<tr>
<td>4.2.2</td>
<td>Servo-Controller and Function Generator</td>
<td>45</td>
</tr>
<tr>
<td>4.3.1</td>
<td>Test Specimen Design Details</td>
<td>47</td>
</tr>
<tr>
<td>4.4.1</td>
<td>Test Specimen Set-up</td>
<td>48</td>
</tr>
<tr>
<td>4.4.2</td>
<td>Test Specimen Instrumentation</td>
<td>49</td>
</tr>
<tr>
<td>4.4.3</td>
<td>Actuator Load Cell and Beam LVDT</td>
<td>51</td>
</tr>
</tbody>
</table>
Figure 6.3.2 Sensitivity Analysis of Model Parameters in Linear Range......................... 83
Figure 6.3.3 Frame Model including Serial Spring .................................................. 85
Figure 6.3.4 Idealized Serial Spring Model............................................................. 85
Figure 6.3.5 Load-Displacement Curve (at Beam) for Serial Spring Model............... 86
Figure 6.3.6 Load-Displacement Curve (at Braces) for Serial Spring Model .......... 87
Figure 6.3.7 Load-Displacement Curve for Reduced Displacement Amplitude Model... 88
Figure 6.4.1 SAP2000 Model Frame Geometry...................................................... 90
Figure 6.4.2 SAP2000 Plastic Hinge Properties...................................................... 91
Figure 6.4.3 Load-Displacement Pushover Curve for Simplified Model................... 92
Figure 7.2.1 Typical Floor Framing Plan................................................................. 94
Figure 7.2.2 Braced Frame Elevation A-A ............................................................... 94
Figure 7.3.1 Bilinear Spring Properties in VSC Chevron Braced Frame Model....... 95
Figure 7.4.1 Conventional Chevron Braced Frame Final Deformed Shape .......... 97
Figure 7.4.2 Conventional Chevron Braced Frame Pushover Curve .................... 97
Figure 7.4.3 VSC Chevron Braced Frame Final Deformed Shape ......................... 99
Figure 7.4.4 VSC Chevron Braced Frame Pushover Curve .................................. 99
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CHAPTER 1
INTRODUCTION

1.1 Background

It is necessary to design structures with adequate stiffness and strength, as well as ductility and energy dissipation capacity, to resist earthquake-induced lateral force demands. Seismic design alternatives are many, and rely on a number of different structural systems and building materials to achieve various levels of performance. Widely used systems include: concrete shear walls, wood frame shear walls, steel moment resisting frames and steel braced frames. Base-isolation systems and energy dissipation elements are also becoming more popular in reducing the seismic force demand on structures. This variety in lateral load resisting systems enables engineers to consider a number of options for seismic design and select an appropriate system based on the governing parameters, be it the seismicity of the region, economic constraints or the overall integration into the building design.

The steel concentrically braced frame (CBF) is a common lateral load resisting system in high-risk seismic areas because of its cost-effectiveness, ease of construction and high stiffness to control the drift under wind loading and during moderate earthquakes. During very severe earthquake-induced shaking, however, CBFs may experience brittle failures at the connections or otherwise result in a slack system when braces yield, leading to a whipping effect with large accelerations. One special subclass of CBFs are chevron braced frames, which consist of two braces forming an inverted V-shape and meeting the underside of the upper storey floor beam at mid-span. Conventional chevron braced frames have an inherent weakness when used in seismic design due to this brace geometry. Once the compression brace buckles, only the tension member will carry any additional storey shear. This will transfer a net vertical force to the floor beam, which may cause a plastic hinge in the beam and thus compromise the gravity load resisting system (Figure 1.1.1). Poor performance of chevron systems in past earthquakes and as observed in laboratory testing has lead to strict design rules limiting chevron braced
frames to the class of nominally ductile concentrically braced frames in the Canadian steel design code CAN/CSA S16.1 [CSA, 1994]. This implies a significant penalty for such systems, which prompted further research into the complex inelastic behaviour of chevron braced frames.

The following sections of this chapter present an introduction to the seismic performance of CBFs, focusing specifically on chevron braced frames, and compare this system to other types of lateral load resisting frames. The historical performance of CBFs and lessons learned about the structural performance of these systems will be highlighted, followed by a brief summary of current design guidelines in the Canadian standards. Finally, the objectives and scope of the study reported here will be discussed.

1.2 Seismic Performance of Concentrically Braced Frames (CBFs)

Lateral load resisting systems are required in every building to resist horizontal forces, static or dynamic, due to wind pressure or seismic accelerations. For steel structures, moment resisting frames (MRFs) and CBFs are two very popular systems, each with their own associated costs and benefits.
In terms of strength and ductility, MRFs have very good performance. However, for taller buildings, their relatively high flexibility makes it uneconomical to design for the necessary stiffness to meet drift requirements [Mazzolani et al., 1995]. In addition, moment connections are much more costly than simple brace connections because of the special detailing required for high ductility as well as the considerable amount of welding involved. Welded connections, especially those done on-site, are also more susceptible to fracture due to imperfections in the welds. Due to their superior energy absorbing characteristics, MRFs were the preferred system in the United States in high-risk seismic zones. The widespread damage to welded MRF connections during the 1994 Northridge earthquake in California severely shattered this belief, which precipitated an extensive research and development project to address the design, detailing, fabrication and inspection of these connections [SAC, 1995].

Compared to most other systems, CBFs have excellent strength and stiffness and easily satisfy rigidity requirements, but offer much less ductility to a structure than MRFs. CBFs are commonly used in low-, medium- and high-rise construction and can be designed in a number of bracing configurations, including cross (X-) bracing, diagonal bracing, K-bracing, V-bracing and chevron (inverted V-) bracing (Figure 1.2.1). When comparing these bracing alternatives, chevron braces are one of the most economical in terms of fabrication and erection costs versus structural effectiveness [Tremblay and Robert, 2000], and the most accommodating in terms of flexibility for locating door and window openings.

![Typical Concentric Bracing Configurations](image)

(a) X-bracing  (b) Diagonal bracing  (c) K-bracing  (d) V-bracing  (e) Chevron bracing

Figure 1.2.1 Typical Concentric Bracing Configurations
Nevertheless, just as the geometry of the chevron bracing system makes it advantageous to use, it also creates some inherent drawbacks to the system. Conventional chevron braced frames consist of two braces forming an inverted V-shape and meeting the underside of the upper storey floor beam at mid-span (Figure 1.2.2). Whereas the compression and tension legs initially carry equal loads, under severe lateral load conditions the compression brace in the system will buckle, resulting in a significant loss of member strength and stiffness. Since the tension brace remains intact, it will continue to attract more axial load, creating a vertical load imbalance in the bracing system, which results in a net downward force on the beam above. This concentrated load at the mid-span of the beam has the potential to cause yielding and plastic hinge formation in the beam. This is a particularly undesirable failure mode, since it compromises the gravity load resisting system and could lead to partial or full collapse of the building.

Figure 1.2.2 Conventional Chevron Braced Frame

1.2.1 Past Performance of CBFs

Damage to CBFs and chevron braced frames in past earthquakes has prompted further research in this area. By examining significant earthquakes over the past thirty years that
have affected steel structures, one can follow the types of changes that have occurred in building codes and modern steel design practices.

The San Fernando Earthquake of February 9, 1971 occurred on the San Fernando fault zone in southern California, an area of thrust faulting that sustained a total surface rupture of about 19 km and maximum slip of up to 2 m. In response to the effects of this earthquake, building codes became more stringent and legislation was passed to prohibit the location of structures for human occupancy across such active faults. Several medical facilities, including the Olive View Community Hospital and Foothill Medical Center, suffered severe damage. Using these buildings as an example, the damage that was observed after the earthquake will help to illustrate the vulnerability of such important structures and demonstrate the performance of two different concentric bracing systems.

The Foothill Medical Center is a two-storey steel framed structure with a plan area of about 61 m by 20 m (200 ft by 65 ft). The lateral load resisting system consisted of MRFs in the longitudinal direction and diagonal bracing on two baylines in the transverse direction of the structure. The building was designed according to the 1962 Los Angeles City Building Code and seismic forces governed the lateral system design.

After the earthquake, major structural damage was identified along one line of the diagonal bracing [Degenkolb and Wyllie, 1973]. It was discovered that the first-floor bracing at one column had been fabricated and installed so that the axis of the brace was approximately 0.3 m (12 in) above the column base plate, instead of intersecting the column at the top of the base plate as designed and detailed (Figure 1.2.3). It seemed that the failure in the bracing had been initiated by yielding and slight bending of the wide-flange column due to the eccentric bracing connection. Once a small deflection of the column took place, it resulted in a significant redistribution of loading and the transfer of forces entirely to the other bracing diagonal. These sizeable axial forces in the diagonal caused shearing of the bolts connecting the first-floor bracing to the second-floor beam. Subsequent to the bolt failure, only one diagonal remained to carry the entire lateral load, in both tension and compression, in the frame. In the end, the column suffered severe
distortion and the column base plate was bent from the compression portion of the column bending. It was also noted that other bracing connection details throughout the structure were modified or omitted, leading to minor instances of further damage to the bracing systems. The second line of transverse bracing also sustained slight damage, but no signs of failure were observed. The longitudinal moment frames performed satisfactorily in the earthquake.

![Image of structural damage](image)

**Figure 1.2.3 Foothill Medical Center: Damage after San Fernando Earthquake**

[Steinbrugge, 1971]

Based on the seismic performance of this building, improvements to current design practices were recommended. It was proposed that the design engineer be required to make field inspections to ensure that the intent of the design is carried out during construction. It was also noted that more care is needed to adequately resist member forces in truss-like connections and to provide a suitable load path for major stress transfer throughout the structure to the foundations. In addition, it was recommended that design codes be more specific as to the amount of compression permitted in bracing members, with particular emphasis that the tension diagonal not be considered sufficient to support the compression diagonal.

Another medical facility of interest was the Olive View Hospital, Heating and Refrigeration Plant. The powerplant was a one-storey building with a partial mezzanine
floor and overall dimensions of about 27 m by 38 m (87.5 ft by 124 ft). The lateral load resisting system consisted of exterior, non-bearing reinforced concrete block walls on the south, east and west ends of the building and diagonal steel strap bracing on the north wall. The irregularity in the lateral load resisting system was attributed to provisions made for future expansion of the structure. The building was designed and detailed to meet the requirements of the 1965 Los Angeles County Building Code.

The damage to the structure initiated with the yielding and stretching of the diagonal straps in the north wall [Johnston and Strand, 1973]. The elongation of the bracing allowed considerable deformations to occur before the straps became effective in subsequent load reversal cycles (Figure 1.2.4). Once the north wall bracing became ineffective, additional lateral loads as well as high torsional loads were induced into the roof decking and side shear walls. It is estimated that this additional loading on the east and west wall reinforcing exceeded the design loads by 250 percent.

![Figure 1.2.4 Olive View Hospital: Damage after San Fernando Earthquake](Steinbrugge, 1971)

Upon review of the damage to the facility, recommendations were made to recognize that the design requirements in the building code serve as a minimum standard for engineering practice, and that it remains the responsibility of the engineer to exercise his or her sound judgement in the final design. In addition, it was noted that the code
considers regular building shape configurations and consistent lateral load resisting systems in its guidelines, and recommended that any building irregularities be minimized or otherwise given special consideration in design. A call was also made for more realistic analysis methods than the “static load equivalent” method, which was the current standard in engineering practice. For steel tension-only bracing in particular, it was noted that the bracing must be designed to accommodate the inelastic deformations of the members as well as the impact from load reversal cycles.

On September 19, 1985, the Michoacan earthquake rocked Mexico City, producing extreme damage to structures that caused thousands of injuries and fatalities. Following design recommendations in seismic codes, highly redundant MRFs were by far the most common steel buildings constructed in Mexico in the three decades prior to this earthquake. Construction typically consisted of welded box columns and trussed beams or girders, with almost every beam-column connection as moment resisting. These buildings performed well in the earthquake and exhibited high levels of ductility, with only a few cases of structural damage reported [Yanev et al., 1991]. The second most common steel building type was the MRF with braced bays. Buildings with these lateral systems had more problems during the earthquake, with the Pino Suarez complex accounting for all reported failures of this system.

The Pino Suarez complex consisted of five high-rise buildings, all of which were steel MRFs with concentric bracing. Three buildings were 21-storeys high and two were 14-storeys high. The moment frames were of typical construction, with welded box columns and trussed beams built up from angles. One line of longitudinal V-bracing was provided in the stairwell bay, and two lines of transverse X-bracing were provided on either side of the stairwell bay, in each building. During the earthquake, one of the 21-storey towers collapsed onto a 14-storey tower, while the remaining 21-storey buildings were left standing but severely damaged.

The failures occurred due to very large axial forces in the exterior columns of the braced bay, which caused yielding in the overloaded columns and local inelastic buckling of the
plates that made up the box columns (Figure 1.2.5). Once these columns in the fourth storey had lost most of their load-carrying capacity, the X-bracing in the storey buckled and all the gravity and seismic overturning loads were transferred to the adjacent columns. Eventually, all exterior columns in the fourth storey failed, due to overloading in compression or bending, and created a collapse mechanism for the structure [Osteraas and Krawinkler, 1989]. The unexpectedly large axial forces that failed the columns were not accounted for in the design of the building. It appears that during the earthquake, the bracing system was capable of resisting storey shears several times higher than the code design levels, and this bracing overstrength created larger than anticipated axial forces in the columns [Yanev et al., 1991].

Figure 1.2.5 Pino Suarez Complex: Column Failure after Mexico City Earthquake

[Osteraas and Krawinkler, 1989]

These findings had a very significant impact on the design of braced frames and specific provisions were included in the subsequent (1988) edition of the Uniform Building Code
(UBC) to address the issue of brace overstrength and prevent accidental overloading of columns in braced bays.

The Northridge earthquake struck the Los Angeles region in the state of California on January 17, 1994, and, once again, the densely populated area of the San Fernando Valley was hit with a moderate, but extremely damaging, earthquake. Because of the large number of new, seismically designed buildings in this developed region of California, the event served as an interesting opportunity to observe the seismic performance of a variety of modern structures.

There was widespread damage to a number of buildings throughout the area, new and old, ranging from non-structural damage to total collapse. Undoubtedly, the message from the observed damage due to the Northridge earthquake was the vulnerability of modern structures, particularly steel MRF buildings. In the days and months that followed the earthquake, it was discovered that there were a significant number of failures of welded moment connections, which occurred at the bottom flanges of beam-to-column connections in most cases, and in isolated cases at the upper flange as well [Bertero et al., 1994]. Although some of these connection failures had not yet been observed in buildings following a major earthquake, the type of damage to the beam-to-column connections was not surprising to some practicing engineers and researchers, as similar behaviour had been detected during experimental testing conducted in research laboratories. However, the serious damage to more than 150 MRF structures initiated an extensive research program to investigate this phenomenon further, and provide strict guidelines for the design, detailing, fabrication and inspection of these connections.

Some modern braced frame structures also suffered damage, and many instances of overall and local buckling were observed in tension-compression bracing systems. The overall buckling phenomenon is an accepted form of deformation and energy dissipation in brace elements, and only becomes a problem when local buckling causes cracking in the regions of high local strains. Under repeated cyclic strain reversals, the cracks lead to fracture of the steel. Many examples of premature fracture of this type were observed in
rectangular and square tubular braces after the Northridge earthquake, as these cold-formed tubular cross-sections have very high residual stresses at the corners where cracks tend to form [Bertero et al., 1994]. Again, research in this area had previously detected these failure modes for tubular bracing members; however, changes had not yet been adopted into the current building code for CBFs.

The Hyogo-ken Nanbu earthquake (also referred to as the Kobe earthquake) occurred on January 17, 1995 in Japan, producing widespread damage to the urban areas of Kobe city. Similar to the Northridge earthquake in the United States exactly one year before, the fault that caused this earthquake ruptured beneath a highly developed, densely populated area and had devastating effects on its natural and built environment. The duration of strong shaking was relatively short in this area, but the magnitude of peak accelerations and strong pulses were large enough to severely damage many structures in the city. Reports indicated that a total of 3406 buildings sustained some level of damage, 1247 of which being steel framed structures [Tremblay et al., 1996]. This is an unusually high percentage of steel structures to have been affected by the earthquake, and further investigation showed that the damage involved mostly older buildings but also a large number of modern steel buildings.

Much of the older building stock in Japan is composed of low- to medium-rise braced steel frames, many of which were built with light-gauge steel construction. The design of modern buildings used superior building materials, fabrication techniques and more advanced seismic design provisions. In small housing complexes and commercial buildings, prefabricated light-gauge steel structures are very common. The construction consists of metal wall panels with thin rod tension-only braces that are installed and pretensioned, with the cross-bracing typically found in every bay [Tremblay et al., 1996]. In most other steel buildings, common lateral load resisting systems include MRFs, CBFs or a combination of both. Cross-bracing and chevron bracing configurations of CBFs are the most common, and are the dominant building systems for elevated parking structures throughout Japan. Tension-only bracing is still used in some newer structures with highly
redundant, multiple bay bracing arrangements. The beams and columns of these structures are typically moment connected, as well.

The performance of steel buildings during the Hyogo-ken Nanbu earthquake was evaluated by a number of expert groups for months following the event. The subsequent damage assessment is based on reports by the Canadian Association for Earthquake Engineering reconnaissance team and the Architectural Institute of Japan [Tremblay et al., 1996]. It was found that, in unbraced frames, the columns suffered the most damage, while in braced frames, braces were the most frequently damaged elements. CBFs suffered damage levels ranging from minor to severe, but the damage to bracing elements was similar to that observed in previous earthquakes in both North America and Japan. The older, light-gauge steel structures experienced a considerable amount of damage, with premature yielding of columns and tension failure of braces. In prefabricated housing structures, there was extensive yielding of bracing members; however, in most cases, the redundancy of the system left enough braces intact to keep the structure from collapsing. In CBFs, the damage appeared more severe in frames with smaller brace cross-sections, which also strongly correlated with the higher age of buildings. Slender braces experienced stretching and slackness due to inelastic yielding. Many connections suffered non-ductile fracture at bolt holes because the connections could not sustain the full brace yield load, in addition to the impact forces developed as braces straightened under tension loading. The more modern tension-compression bracing systems also suffered extensive damage, with severe inelastic deformations of buckled bracing members. Typical failure modes included the low-cycle fatigue fracture of braces at plastic hinge locations, net section fracture of bolted brace connections, bolt shearing and failure of welded connections between gusset plates and the surrounding frame (Figure 1.2.6). Beams were also seriously damaged at the middle of split X-braces or above chevron braces, due to cyclic load reversals in the bracing members.
The number of frames that suffered extensive damage but did not collapse was in part due to the redundancy in the design of Japanese structures. Not only did most braced frames also include fully rigid beam-to-column connections in addition to the bracing, most structures also included a larger number of braced bays than would be typical in North American construction.

A number of valuable lessons were learned from the earthquake, and many strong recommendations were made for design practice in high-risk seismic regions [Tremblay et al., 1996]. With Kobe city located so near to the fault rupture location, the event once again demonstrated the destructive potential of large earthquakes occurring close to major cities. Recommendations were made to learn more about near-field ground motions and their effects on neighbouring structures; however, due to the inherent uncertainty associated with the prediction of ground motions and building response, the emphasis
was turned to the ductile design of structures, minimization of building irregularities and utilization of redundancy in building systems. In addition, the Hyogo-ken Nanbu earthquake drew further attention to flaws in detailing and fabrication practices in steel design. In CBFs, the damage reported reflected the importance of proper connection design. Recommendations were made to examine the appropriateness of considering potential brace overstrength in connection design, as is done in Japan, in addition to the current CAN/CSA-S16.1 provisions in Canada. Gusset plate failures in compression bracing systems also suggested that gusset plates be designed with the anticipated brace yield load, but also the compression load, in mind. Finally, the performance of tension-only braced frames in Kobe indicated that satisfactory behaviour was attainable with well-designed connections and extensive system redundancy.

1.2.2 Structural Performance of CBFs

A number of key elements have been identified in determining the level of performance of CBFs, both from observations made in laboratory testing as well as damage assessments following major seismic events. Some of these important aspects of structural performance will be highlighted, with special emphasis on critical details of chevron braced frames.

In all configurations that utilize brace elements in both tension and compression, brace buckling behaviour is a key point of interest. Braces will have tendencies to buckle in different arrangements based on boundary conditions, and considerations must be made for in-plane or out-of-plane buckling behaviour. One key parameter identified as influencing buckling characteristics is brace slenderness, and recommendations have been made as to appropriate ranges for slenderness values in CBF design [Khatib and Mahin, 1987; Tremblay, 2000].

In seismic applications, the post-bucking performance of brace elements is of particular importance to the overall behaviour of the system. The appropriate detailing of bracing members and connections is critical, as the residual stiffness and strength of the braces is
a key factor in the ductility and energy absorbing capabilities of the system. Detailing must include allowances for plastic hinges to form in gussets plates at the end of braces such that the anticipated large inelastic post-buckling deformations can occur. Severe local buckling at the brace mid-span plastic hinge location often leads to early fracture and low-cycle fatigue problems in cold-formed rectangular tube braces. This is due to the concentration of strain at the corners of the compression flange and web [Liu and Goel, 1988]. Controlling the brace member width-thickness ratio has been cited as being critical to preventing early fracture [Tang and Goel, 1987], and the use of concrete infill has been recommended as it was found to increase the fracture life by up to 300% [Lee and Goel, 1987].

When the ductility demand on a structure cannot be met with bracing alone, a dual system combining MRFs and CBFs has been used to take advantage of the benefits of both systems. Dual systems attempt to improve system redundancy, as the inability of braced frames to meet ductility demands often results in soft storey formation and potential failure of the lateral load resisting system. Thus, force redistribution in dual systems is of key importance, and numerous experimental and analytical studies have addressed these issues [Yamanouchi et al., 1989; Roeder, 1989; Fukuta et al., 1989; Lee and Lu, 1989] and resulted in design code recommendations [Bertero et al., 1989].

For conventional chevron braced frames in particular, consideration must be given to the beam design at the convergence of the brace members. Typically, the force imbalance in the braces immediately following buckling of the compression member is applied as a downward force on the beam as the load in the tension brace continues to develop. The resulting shear force acting on the beam may lead to plastic hinge formation at the mid-span of the member, compromising the gravity load resisting system. Unless there is sufficient redundancy in the design, the beam damage following a large lateral force demand can lead to progressive collapse of the structure. For these reasons, the protection of the beam in chevron bracing systems has become a key focus of research efforts, and various design concepts for the beam have been proposed [Remennikov and Walpole, 1998; Tremblay and Robert, 2000].
1.2.3 Design Requirements for CBFs

The current Canadian standard for Limit States Design of Steel Structures, CAN/CSA-S16.1-94, classifies the chevron brace system as a concentrically braced frame with nominal ductility (NDBF), and assigns this category of lateral load resisting systems a force modification factor of 2. This limitation is largely due to the susceptibility of chevron brace systems to damage to the beam and gravity resisting system of the structure. There is an additional limitation on the number of storeys in buildings with chevron braced frames, based on studies demonstrating declining performance with increasing overall building height [Tremblay and Robert, 2000].

Another major factor governing the design of CBFs is the capacity design philosophy that has been adopted by the Canadian code since 1989. In compliance with the principle of capacity design, the code requires that end connections of braces be designed for the full tensile capacity of the selected brace element, i.e. \( A_g F_y \) in which \( A_g \) is the gross cross-sectional area and \( F_y \) is the yield strength. In many cases, this connection force is much larger than the design force of the member, as concentric braces are typically selected based on the compression capacity of the member. This has significant implications on the cost of a structure in terms of increased material and labour costs for the structural steel system, as well as increased foundation sizes [Rezai et al., 2000].

1.3 Objectives of Research

In response to the need for cost-effective design alternatives in the British Columbia (BC) region, a modified chevron braced frame, designed to mitigate problems associated with conventional chevron systems, was developed by design engineers at the consulting firm Fast & Epp in Vancouver, BC. A prototype system was designed for further testing and investigation by researchers at the University of British Columbia. The innovative detail incorporates a vertical slotted connection (VSC) at the junction between the inverted V-
braces and the beam (Figure 1.3.1), designed to restrict vertical load transfer to the floor beam above and limit brace forces to the compression capacity of the brace members.

To evaluate the VSC chevron brace system, a research program was conducted at the University of British Columbia. The objectives of the research were:

- To investigate the behaviour of the proposed system under quasi-static cyclic loading conditions, and
- To assess the suitability of the VSC chevron brace system as a lateral load resisting system for high-risk seismic zones.

1.4 Scope of Research

The research described in the following chapters focuses on the behaviour and response of the VSC chevron brace system. The scope of this study includes:
- Experimental testing on two full-scale specimens subjected to quasi-static cyclic loading. One of the specimens consisted of unfilled hollow structural section (HSS) braces, whereas the second specimen had concrete-filled HSS braces.

- An analytical study consisting of the calibration of a non-linear model of the VSC system, using hysteretic data derived from the results of the experimental testing as the basis for evaluation. The exercise also served to provide an assessment of current modelling techniques for complex non-linear brace buckling behaviour using existing commercially available software packages.

- A comparative non-linear static pushover analysis of a typical medium-rise building using conventional chevron braced frames and the new VSC chevron braced frames.

- Design guidelines for practicing engineers for the detailing of VSC chevron braces and analytical modelling of buildings with VSC systems.
CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

The following chapter provides an overview of engineering literature and research on chevron braced frames, along with a discussion of the significance of the current research that is presented in this study on VSC chevron braced frames.

2.2 Past Research

There are a number of examples of past research projects that address the concerns associated with CBFs and chevron bracing systems. Of the wide range of literature available on concentric bracing and that which has been reviewed, the material that will be presented has been separated into two categories, namely (i) studies to identify and improve deficiencies in conventional chevron braced frames, and (ii) studies to evaluate the use of energy dissipation elements within CBF systems.

2.2.1 Conventional Chevron Braced Frames

The focus of many key studies on chevron braced frames has been the behaviour of these systems when subjected to cyclic force demands and key parameters influencing their response. Following these stages of research, alternatives to conventional design methodologies were proposed, based on experimental or analytical findings to support new ideas. Of significance to chevron bracing research are studies that can improve the understanding of the complex non-linear behaviour of the system, as well as studies that aim to improve the overall frame stability through examination of the interaction between the braces and the floor beams above.
Chapter 2

Literature Review

An extensive analytical study to examine the behaviour of concentrically chevron braced frames was done at the University of California at Berkeley [Khatib and Mahin, 1987]. The study was conducted on the exterior frame of a six-storey building model. The building was square in plan, with three 20 ft. bays on each side, and only the exterior frames were assumed to be braced. The investigation focused on different configurations of CBFs and various methods for improving their seismic response. Non-linear quasi-static analyses identified that the key factors controlling inelastic brace behaviour are brace slenderness and relative beam stiffness. The effects of variations in these parameters were studied using dynamic response analyses of inverted V-braced frames of different proportions in the model. The evaluation of the performance of these systems was based on the following response quantities: energy dissipation demand, storey shears, storey drifts and column compression as an indicator of force redistribution to the frame due to brace buckling. Results indicated that the combination of moderately slender braces and flexible beams is the least desirable configuration. The combination of very slender braces with stiff beams provided better hysteretic behaviour, but induced large compressive forces to the columns. This configuration would also contravene the “weak beam, strong column” design philosophy. Stocky braces provided the most favourable response characteristics; however, in general, the prevention of local buckling remained an issue for the braces.

Also investigated were potential improvements to CBFs through changes to the structural system, such as different bracing configurations or greater moment frame participation in lateral load sharing. Findings from the analytical parametric studies indicated advantages and disadvantages for each of the alternatives, depending on the circumstances. Recommendations from the study suggested further work be done to investigate the dynamic force redistribution in these modified systems. It was also emphasized that, in all cases, the capacity design approach should be used.

Research was also done in New Zealand on the performance of low-rise inverted V-braced frames with specific emphasis on brace behaviour and interaction with floor beams, as well as deterministic design procedures for these systems [Remennikov and
Walpole, 1998]. Remennikov and Walpole acknowledged that this configuration of CBFs may exhibit undesirable failure modes under severe earthquake loading, but suggested that a suitable seismic design procedure may ensure dependable inelastic seismic performance. Experimental and analytical results from previous studies by others have shown that the behaviour of the inverted V-braced frame depends largely on the ratio of the beam moment resistance to the brace resistance in tension. Therefore, an analytical study was done to assess the performance of three different configurations of a two-storey CBF, each using a different design approach for the beam elements attached to the inverted V-bracing, subjected to near-field type ground motions. Design approaches included those for an ordinary CBF with an inverted V-brace [UBC, 1997], a special CBF with an inverted V-brace [UBC, 1997] and an inverted V-brace designed according to the Heavy Engineering Research Association (HERA) Report [Feeney and Clifton, 1995].

Results of the study showed that the design rules for ordinary CBFs result in a frame incapable of resisting severe ground shaking, with inter-storey drift demands exceeding code limits and potential structural failure. Requirements for special CBFs appeared to be somewhat conservative in these applications, and the HERA Report rules appeared to produce frames that require excessive structural ductility. According to these findings, the authors proposed a different design methodology, considering 80% of the yield strength of the tensile braces in the design of the beams, and suggested that its implementation would lead to improved seismic performance of inverted V-braced steel frames.

Further to Remennikov and Walpole's studies, researchers at École Polytechnique in Montreal [Tremblay and Robert, 2000] investigated the seismic design of low- and medium-rise chevron braced frames from the standpoint of the current Canadian code. Two different design approaches were presented for multi-storey chevron braced frames. The first method followed current CSA-S16.1 [CSA, 1994] design requirements for chevron bracing, which are limited to the category of CBFs with nominal ductility (NDBF). In this design, the beams must possess enough strength for the initiation of brace buckling, but are not required to meet a minimum flexural capacity to reduce the
Chapter 2

Literature Review

excessive stiffness and strength degradation that is typical in conventional chevron braced frames. In the second method, the beams were sized to also carry the additional loads from the braces after buckling has occurred, with the recommendation that these systems qualify under the category of ductile braced frames (DBFs) because of their improved inelastic response. The seismic loads in DBF structures are 2/3 of those in NDBF structures due to the differences in ductility and, therefore, force modification factors in each category. Three beams with different moment capacities were examined using this approach, which corresponded to the DBF-100, DBF-80 and DBF-60 systems. In these designations, the required beam capacities are expressed as the percentage of the brace tension yield load (A_yF_y) that the beams can develop after buckling of the compression braces.

The analytical study focused on the application of each of these design approaches in 2-, 4-, 8- and 12-storey structures with construction typical for Vancouver, BC. The results of the study compared the seismic performance of each of the structures as well as their steel tonnage requirement, which acts as an indicator of the economy of the system.

The most pronounced finding of the study was that although smaller bracing members are used in the DBF design (due to lower seismic forces), the impact on the overall steel weight is generally minor. This is because the additional material needed for the beams offset any cost savings expected from using a DBF design. However, in terms of seismic performance, the DBF-100 system was superior to the NDBF system and, to a lesser extent, the DBF-80 and DBF-60 systems in terms of storey shear capacity after buckling of the braces. Further research into the dynamic inelastic seismic behaviour of these systems was recommended to better evaluate the cost-performance ratio.

In terms of the specific case of economics of low- and medium-rise construction in Vancouver, BC, this research clearly indicates a need to explore chevron brace systems that protect the beam and gravity load resisting system in a cost-effective way, while maintaining an acceptable level of performance in seismic events.
2.2.2 Energy Dissipation Elements in CBFs

A need has been identified in the Canadian steel industry for researchers and professionals to develop and test structural systems that limit brace forces to the capacity of the energy dissipation elements in the lateral load resisting system. This design methodology would alleviate the need to design connections for the full tensile capacity of brace elements and prevent the propagation of these design forces throughout the rest of the structure. Numerous design alternatives have been proposed that utilize energy dissipation elements in CBFs, either as part of the brace members or linking brace elements to the frame, some of which are described here in further detail.

An experimental study was conducted at the University of British Columbia and École Polytechnique in Montreal to examine the potential for using ductile brace fuse elements to dissipate seismic energy and reduce the force demand on brace connections in steel CBFs [Rezai et al., 2000]. The objective of the research was to determine if there was promise for the development of these controlled fuses for hollow structural section (HSS) braces in the building industry, as members in the structural design and steel industry in British Columbia were looking for ways to combat the large disparities between governing brace capacities and specified connection design forces ($A_g F_y$).

The experimentation was done in three phases, consisting of (i) full-scale tests of fuse details in cross-braced frames, (ii) cyclic tension-compression tests on fuse details alone, and (iii) full-scale tests on preferred fuse details in chevron braced frames. Fuse details that showed the most promise and continued to the final phase of testing were elliptical cut-outs in all four walls of HSS tubes, tapered fuse plates between two halves of HSS brace members and dog-bone shaped fuse plates between two halves of double angle brace members. All details included provisions to prevent global and local buckling of the specimens using internal and external sleeves with appropriately sized HSS tube segments along with additional reinforcement as required. Improvements were made to fuse details based on performance in earlier phases of testing prior to final testing in the
full-scale specimens. The final full-scale testing was conducted using both an increasing stepwise displacement-controlled loading history, as well as a loading history obtained from non-linear dynamic analyses of a typical braced frame building.

Phase three test results showed stable, full hysteresis loops for all specimens with strain hardening response and no strength degradation until fracture occurred. Trends in the fracture life of different specimens indicated increased life with increased fuse length. It was also observed that prevention of global and local buckling is critical to the performance of fuse elements, as specimens found to have inadequate lateral support exhibited poor overall behaviour. It was concluded that the ductile fuse concept was feasible for introduction into CBFs for steel buildings, and suggestions were made for the enhancement of various ductile fuse details along with recommendations for further research into the prediction of fatigue life for fuse elements.

Another form of innovation in CBFs involves the use of slotted bolted connections (SBCs) as inelastic energy dissipators within brace elements through friction resistance during load reversal cycles. Two series of experimental testing were done at San Jose State University (SJSU), the first by Venuti (1976) to determine the feasibility of the SBC detail, and the second by Zsutty (1984) to examine the performance of modifications to the original test configuration [Fitzgerald et al., 1989]. The proposed arrangement of the SBC detail consisted of a gusset plate, two back-to-back channel sections, cover plates and bolts with Solon (Belleville) washers. Braces were designed such that buckling would not occur, as the slip force in the SBC would be lower than the brace critical buckling load.

Although only a limited number of specimens were tested, results indicated that dependable load levels and behaviour are achievable. Load-deformation diagrams obtained from testing indicated stable rectangular hysteresis loops for a large number of inelastic cycles. Further testing was recommended by the researchers to verify the frictional response and deformation behaviour of multiple bolt assemblies or larger bolt sizes.
Further to the work done at SJSU, testing of SBC systems continued at the University of California at Berkeley [Grigorian et al., 1993] with the examination of different contact materials for the friction surfaces. In one arrangement, friction occurred between clean mill scale steel surfaces; in the other, friction was between clean mill scale steel and brass surfaces. The development of SBCs as energy dissipation devices is of interest because other types of devices require costly, specialized manufacturing or installation, whereas SBCs attempt to keep the design, fabrication, material and installation costs significantly lower than for systems with special energy dissipating devices.

Results of testing confirmed the rectangular hysteresis diagrams and nearly elastic-perfectly-plastic behaviour observed in previous testing for the SBCs with brass insert plates. In addition, the slip forces for brass and steel frictional surface SBCs remained relatively constant over the range of interest, whereas the slip forces for steel-to-steel contact surfaces varied significantly. The authors thus recommended that brass frictional surfaces be used in SBCs because of superior energy dissipation and predictable behaviour, and that these systems be considered low-cost alternatives for energy dissipation in the seismic design and retrofit of structures.

From these results, it is evident that one of the main benefits of the SBC braced frame system is that the lateral force resistance of the frame can be well controlled to a given design level. In addition, these resisting forces are independent of the bracing member capacities, so a decrease in slenderness of a bracing member to avoid inelastic buckling does not result in a corresponding increase in tensile or compressive force resistance in the system. In conventional CBFs, any strengthening of bracing members must be accounted for by increasing the capacities of connections and vertical supporting elements in the frame.

Another type of energy dissipation system proposed for use with chevron braced frames provides an additional element between the braces and the beam in the frame, attempting to concentrate the energy dissipation in an element other than the braces. Testing of one
such system was carried out by the Building Research Institute in Japan [Midorikawa et al., 1996] to evaluate the seismic performance of a steel beam energy dissipation (SBED) device incorporated into a chevron frame system. Three single-bay steel frames were fabricated, each with a different type of SBED device located between the girder and inverted-V braces. The devices were all wide-flange beams built up from different grades of steel for the flange and web, namely (i) mild steel for the flange and low-yield-point steel for the web, (ii) mild steel for the flange and super-low-yield-point steel for the web, and (iii) high-strength steel for the flange, super-low-yield-point steel for the upper-half web and mild steel for the lower-half web. Each test frame was subjected to cyclic horizontal loading to study the inelastic behaviour of the frame and performance of the SBED device.

The test results indicated many favourable characteristics of the steel frames with hybrid SBED devices as seismic energy dissipation systems, particularly when compared to non-hybrid section, as shown in previous tests. All test frames showed large deformation capacities and stable hysteretic behaviour without degradation in strength or stiffness for storey drifts up to 4%. The measured storey shear strength was conservative when compared to the calculated strength. The maximum storey shear strength was approximately 1.3 times the calculated value due to the work-hardening effects of the devices during inelastic load reversals.

An example of a similar type of system is the triangular-plate added damping and stiffness (TADAS) device, which was the subject of experimental and analytical research at the National Taiwan University [Tsai et al., 1993]. The TADAS frame system incorporates the use of welded triangular steel plates as ADAS devices at the brace-to-beam connection in chevron braced frames (Figure 2.2.1). A number of researchers have done past studies with X-shaped steel plates in ADAS devices and shown that the concept is suitable for use in buildings to resist seismic forces. However, as bolts were used to join both ends of the steel plates together in such devices, it was found that the stiffness of the device was sensitive to the tightness of bolts and that the stiffness obtained was up to 35% less than predicted by assuming both plate ends were fixed [Whittaker et al.,
The concept of using a welded steel TADAS device was investigated through cyclic load tests of eleven TADAS device specimens, pseudo-dynamic testing of two-storey full-scale steel frame utilizing the TADAS devices and non-linear response spectrum analysis of a single degree-of-freedom TADAS system.

The proposed TADAS device consisted of several triangular plates welded together to a common base plate that was fixed to the testing apparatus. The opposite ends of the triangular-plates were each pin-connected to a horizontal channel through the use of vertical slotted holes, and the channel was attached to a hydraulic actuator from which the lateral loading was applied. The triangular-plate fabrication and welding details followed very strict specifications for connection to the common base plate, while the height to thickness ratios for the triangular-plates were varied for each of the eleven cyclic tests. From the results of the testing, it was found that properly welded TADAS
devices could sustain a large number of load reversal cycles without signs of stiffness or strength degradation, and that the typical rotational capacity of the devices was ±0.25 radians. The effects of strain hardening were evident when large bending deformations in the device were observed.

In the pseudo-dynamic testing, a two-storey frame with TADAS devices between chevron braces and the connecting beams was subjected to several earthquake ground acceleration records with different intensities. It was found that during the inelastic response of the frame, the maximum storey drift angle observed was 0.016 radians and the maximum TADAS rotational demand was only 0.14 radians, which is considerably less than the typical rotational capacity (listed above) found in the cyclic tests. These experimental results were also compared with analytically predicted responses, with satisfactory overall agreement between the two. In these analyses, the TADAS device was modelled with a prismatic beam element using theoretically derived values for elastic stiffness and plastic strength. In terms of the design implications for the device, one of the most favourable characteristics highlighted in the study is that the gravity frame is completely separated from the energy absorbing device by the use of slotted hole in the connection details. In addition, the yielding and damage to the frame can be contained entirely in the TADAS device; therefore, following a capacity design approach, the braces, beams and columns can be designed for the maximum forces in the devices including strain hardening effects.

In the analytical study, the behaviour of the TADAS system was characterized to develop a hysteresis rule for inclusion into a non-linear response analysis program. In the constructing this model, it was first recognized that the horizontal stiffness of the TADAS element (made up of the TADAS device and two braces that support the device), $K_a$, is a function of the lateral stiffness of the braces, $K_b$, and the device stiffness, $K_d$ (2.1). In addition, the elastic stiffness of the overall frame system is a function of the TADAS element stiffness, $K_a$, and the stiffness of the surrounding frame alone, $K_f$ (2.2).
It can be seen from the form of (2.1) and (2.2) that the stiffness of the braces and TADAS device are related to the stiffness of the TADAS element as two springs in series. In contrast, the stiffness of the TADAS element and frame are related to the stiffness of the overall frame system as two springs in parallel. The observed load-deformation responses of each of these components were used as described above to create the force-deformation model along with the tri-linear hysteresis rule for the TADAS overall frame system. From the results of a single degree-of-freedom response analysis, some general trends in results were noted in terms of the system stiffness ratio (SR), where SR is the ratio of $K_a$ to $K_f$. It was found that systems with a large SR value showed a smaller displacement response, but a larger acceleration response. For systems with a short vibration period, the larger the SR value, the lower the total energy input. In contrast, for systems with a medium to long vibration period, the smaller the SR value, the lower the ductility demand on the device.

In summary, many of the energy dissipation systems described in this section show merit for use in practice; however, there are benefits and drawbacks to each system in terms of specialized design, fabrication, materials or installation. In addition, some details that rely heavily on welded connections could be costly to implement when compared to the traditional approach to CBF design, which attempts to minimize reliance on welded connections, particularly those done on-site.

2.3 Significance of Current Research

From the research on conventional chevron braced frame design and energy dissipation elements in CBFs presented above, it is clear that there is a need to develop cost-effective systems to resist seismic force demands on CBF structures. Many newly developed systems and research efforts show merit for their efficiency in meeting these demands,
but there are issues related to the economy of these systems that remain unresolved. Ideally, cost-effective design alternatives should be relatively easy to design and fabricate, incorporate standard materials and require no special installation. Additionally, a key to the successful overall performance of chevron braced frames is the protection of the gravity load resisting system, while providing adequate lateral force resistance. Finally, effective modifications to conventional chevron braced frames must meet the specific needs of the end users, namely the building owners, practicing engineers and local steel industry. In this case, the low- and medium-rise construction market in BC is looking for ways to alleviate design difficulties with the requirements for bracing connections in the current Canadian standard for steel design (described in Section 1.2.3).

For these reasons, an innovative design solution that would help mitigate problems associated with conventional steel chevron braced frames was investigated, which incorporates a vertical slotted connection at the junction between the inverted V-braces and the beam. The concept behind this design detail is that the system limits the brace axial forces to the compression capacity of the brace members and restricts vertical load transfer to the floor beam above. The ductility and energy absorbing capacity of the system is developed as the braces buckle in compression and undergo inelastic deformations during subsequent load reversal cycles, thereby effectively dissipating the energy input from an earthquake excitation.

This system addresses the needs identified above; however, the seismic performance of the system remains to be identified. Through a combination of physical testing and analytical modelling, the behaviour and feasibility of the VSC chevron system will be assessed.
CHAPTER 3
SPECIMEN MATERIAL TESTING

3.1 Introduction

Material testing was performed on samples of all steel members that were part of the chevron bracing testing assembly, which included the square HSS tube used for the brace members and two sizes of gusset plate used for the top and bottom brace connections. Two types of standard tests were performed on the material samples; tension tests were done for all test coupons to determine general material properties, and compression tests were done for square HSS tube columns to determine buckling characteristics of the specimens.

3.2 Tension Tests

Tension tests were conducted on test coupons according to the appropriate American Society for Testing and Materials standards [ASTM, 2001] prior to full-scale testing of the specimens.

3.2.1 Description of Test Set-up

The apparatus used for the tests was the 400 kip (1780 kN) Baldwin Universal Testing Machine, located in the Civil Engineering Structures Laboratory at the University of British Columbia (Figure 3.2.1). Tests were performed at the medium range of the machine, with up to 80 kip (356 kN) loading capacity. Test coupons were machined from the same batch of steel as the fabricated test specimens, and mill certificates were obtained for all materials tested. A series of tension tests were performed on each of the following steel material samples: HSS tube wall segments, HSS tube corner segments, 12.7 mm (0.5 in) plate, and 19.0 mm (0.75 in) plate. A 50 mm gauge length was used on each test coupon to measure specimen elongation during testing (Figure 3.2.2).
Figure 3.2.1 Baldwin Universal Testing Machine

Figure 3.2.2 Material Testing Coupons for Tension Tests

(a) HSS tube wall  (b) HSS tube corner  (c) 12.7 mm plate  (d) 19.0 mm plate

Note: All dimensions in mm.
Standard grips were used for all of the test coupons with the exception of the HSS tube corner specimens. Due to the curvature of the cross-section, a special set of grips was fabricated to match the rounded profile of the specimen and provide better surface contact with the interface between the specimen and the testing machine (Figure 3.2.3).

![Custom Grips for HSS Tube Corner Test Coupon](image)

**Figure 3.2.3 Custom Grips for HSS Tube Corner Test Coupon**

### 3.2.2 Instrumentation

Elongation over the gauge length of the specimen was measured using one Linear Variable Differential Transformer (LVDT) that was clamped to the specimen (Figure 3.2.4). (Complete instrumentation details can be found in Appendix A.) Applied forces on the specimen were measured with the Baldwin Machine load cell. These two channels of data from the tests were collected using a computer controlled data acquisition system (Figure 3.2.5).

### 3.2.3 Test Protocol

Measurements of all test coupons were taken prior to testing, using digital callipers to determine most dimensions. The speed of testing was approximately 2 MPa/s up to
yielding of the specimen, and increased to 0.06 m/m/min until tensile failure of the specimen, which is within the allowable range specified by the testing standard.

Figure 3.2.4 Test Coupon with Attached LVDT
3.2.4 Test Results

Three tests were conducted for each material sample, and results were averaged to determine the actual material properties. The determination of yield strength was done using the 0.2% strain offset method. A summary of the material test results can be found in Table 3.2.1, Table 3.2.2, Table 3.2.3 and Table 3.2.4. Typical stress-strain curves for all the material samples can be found in Figure 3.2.6, Figure 3.2.7, Figure 3.2.8 and Figure 3.2.9.

Table 3.2.1 Material Properties for HSS Tube Wall Test Coupons

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Specified (MPa)</th>
<th>Test 1 (MPa)</th>
<th>Test 2 (MPa)</th>
<th>Test 3 (MPa)</th>
<th>Test Average (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Strength, Fy</td>
<td>350</td>
<td>422</td>
<td>423</td>
<td>432</td>
<td>426</td>
</tr>
<tr>
<td>Ultimate Strength, Fu</td>
<td>450 - 650</td>
<td>515</td>
<td>509</td>
<td>517</td>
<td>513</td>
</tr>
</tbody>
</table>
### Table 3.2.2 Material Properties for HSS Tube Corner Test Coupons

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Specified (MPa)</th>
<th>Test 1 (MPa)</th>
<th>Test 2 (MPa)</th>
<th>Test 3 (MPa)</th>
<th>Test Average (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Strength, Fy</td>
<td>350</td>
<td>583</td>
<td>597</td>
<td>559</td>
<td>580</td>
</tr>
<tr>
<td>Ultimate Strength, Fu</td>
<td>450 - 650</td>
<td>632</td>
<td>655</td>
<td>617</td>
<td>634</td>
</tr>
</tbody>
</table>

### Table 3.2.3 Material Properties for 12.7 mm Plate Test Coupons

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Specified (MPa)</th>
<th>Test 1 (MPa)</th>
<th>Test 2 (MPa)</th>
<th>Test 3 (MPa)</th>
<th>Test Average (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Strength, Fy</td>
<td>300</td>
<td>362</td>
<td>362</td>
<td>370</td>
<td>365</td>
</tr>
<tr>
<td>Ultimate Strength, Fu</td>
<td>450 - 620</td>
<td>516</td>
<td>518</td>
<td>517</td>
<td>517</td>
</tr>
</tbody>
</table>

### Table 3.2.4 Material Properties for 19.0 mm Plate Test Coupons

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Specified (MPa)</th>
<th>Test 1 (MPa)</th>
<th>Test 2 (MPa)</th>
<th>Test 3 (MPa)</th>
<th>Test Average (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Strength, Fy</td>
<td>350</td>
<td>406</td>
<td>403</td>
<td>404</td>
<td>405</td>
</tr>
<tr>
<td>Ultimate Strength, Fu</td>
<td>450 - 650</td>
<td>573</td>
<td>570</td>
<td>573</td>
<td>572</td>
</tr>
</tbody>
</table>

Figure 3.2.6 Typical Stress-Strain Curve for HSS Tube Wall Test Coupon
Figure 3.2.7 Typical Stress-Strain Curve for HSS Tube Corner Test Coupon

Figure 3.2.8 Typical Stress-Strain Curve for 12.7 mm Plate Test Coupon
The general shapes of the stress-strain curves for the HSS material samples were typical for cold-formed steel sections, with relatively undefined yield points and moderate inelastic strain ranges. The difference in material behaviour for the two types of HSS tube samples clearly distinguished distinct residual stress conditions in the wall and corner segments due to the cold-forming process. As one would expect, the strength of the corner segments was higher, but the behaviour much more brittle. For the entire tube cross-section, the average yield strength of the HSS brace was calculated to be 455 MPa, based on weighting test results according to cross-sectional area proportioning of the tube walls and corners.

The flat plate material samples showed stress-strain curves characteristic of hot-rolled steel sections, and all results were within an expected range of values.

### 3.3 Stub-Column Compression Tests

Compression tests were conducted on HSS tube samples according to the standard Stub-Column Test Procedures [Galambos, 1988]. The test is designed for cold-formed steel
sections with relatively thin-walled plate elements and its goals are to observe the effects of local buckling and cold-forming on column strength and performance.

3.3.1 Description of Test Set-up

The Baldwin Universal Testing Machine (Figure 3.2.1) was used for the compression tests, utilizing the high range of the machine, which could provide up to 400 kip (1780 kN) loading capacity. Material for the stub-column test specimens was from the same batch of steel as the chevron braces of the fabricated test specimen, and mill certificates were obtained for the steel. Two specimens were cut from the 89 mm x 89 mm x 4.8 mm HSS tube sections, and milled to a length of 300 mm (Figure 3.3.1). Each specimen was set in the testing machine between flat bearing plates, to ensure uniform loading across the full cross-section, and whitewashed before testing to aid in the visual detection of yielding and local buckling of the member.
3.3.2 Instrumentation

Strain gauges were mounted at six locations at mid-length of the stub-column specimens to measure strains during testing (Figure 3.3.2). Two LVDTs were also set-up with magnetic bases on the testing machine to measure the displacement of the loading platform during compression loading (Figure 3.3.3). These eight channels of data, along with loading data from the Baldwin Testing Machine, were connected to an 8-channel data acquisition system and computer for data collection and storage (Figure 3.2.5).
3.3.3 Test Protocol

The loading during testing was continuous, at a rate of approximately 1 MPa/min until yielding, then between 1 and 2 mm/min until the end of the test, in order to capture as many experimental data points as possible in the elastic and inelastic regions of the stress-strain curve. The test was continued until one of the end conditions specified by the standard had been met. For these tests, it was when the load dropped to approximately half of the predicted yield level load.

3.3.4 Test Results

Two stub-column tests were conducted and results were averaged to determine material properties. The determination of yield strength was done using the 0.2% strain offset method. A summary of the stub-column test results can be found in Table 3.3.1 and a typical experimental stress-strain curve is shown in Figure 3.3.4.

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Specified</th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test Average</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(MPa)</td>
<td>(MPa)</td>
<td>(MPa)</td>
<td>(MPa)</td>
</tr>
<tr>
<td>Yield Strength, Fy</td>
<td>350</td>
<td>457</td>
<td>458</td>
<td>458</td>
</tr>
<tr>
<td>Ultimate Strength, Fu</td>
<td>450 - 650</td>
<td>524</td>
<td>525</td>
<td>524</td>
</tr>
</tbody>
</table>
Figure 3.3.4 Typical Stress-Strain Curve for HSS Stub-Column

The measured yield strength for the HSS stub-columns was in good agreement with the value determined from the tension tests. The onset of local buckling in the column specimen is denoted by the point on the stress-strain curve where the load peaks followed by an immediate reduction in strength capacity. The buckled shapes were consistent on both specimens, and consisted of outward bulges near the bottom of the column on opposite tube walls, and inward bulges on the adjacent walls (Figure 3.3.5). There were also slight signs of a buckling pattern along the longitudinal direction of the specimen, as the buckling shapes appeared to continue, in a much less pronounced pattern, toward the top of the column.
Figure 3.3.5 Failed HSS Stub-Column Specimens
CHAPTER 4
QUASI-STATIC CYCLIC TESTING

4.1 Introduction

Full-scale tests were conducted on two VSC chevron brace specimens to study their response to quasi-static cyclic loading conditions and energy dissipation capacity. The first specimen was tested with unfilled hollow structural section (HSS) tube braces; the second specimen had concrete-filled HSS tube braces.

4.2 Description of Testing Facility and Equipment

Quasi-static cyclic testing was conducted in the Civil Engineering Structures Laboratory at the University of British Columbia. The testing was done with the specimen mounted in a heavy steel reaction frame, large enough to accommodate a single-bay braced frame with realistic storey height and bay width. The testing frame was anchored to the laboratory strong floor and was stiff enough to prevent any interaction with the forced response of the specimen being tested (Figure 4.2.1). The hydraulic actuator used to load the specimen had a capacity of 450 kN (100 kip) and a stroke of ±30 cm (±12 in). The actuator displacement was controlled by an MTS servo-controller and the appropriate time versus position signals were provided by an analog function generator (Figure 4.2.2).
Figure 4.2.1 Permanent Testing Frame in Civil Engineering Structures Laboratory

Figure 4.2.2 Servo-Controller and Function Generator
4.3 Specimen Design

The steel specimens were designed by Fast & Epp Structural Engineers of Vancouver, BC, according to current Canadian code provisions for bracing members [CSA, 1994] and dimensioned to the maximum capacity of the research facility testing frame. The specimen dimensions were 3.69 m in height and 3.51 m in width, as shown in Figure 4.3.1. The brace elements were HSS square tubes (89 mm x 89 mm x 4.8 mm) with specified yield strength of 350 MPa. (Refer to Chapter 3 for actual material properties). Braces were slotted at the ends and welded to gusset plates to facilitate connection to the frame. At the bottom of the assembly, 12.7 mm gusset plates were welded to base plates with bolt-hole patterns to match the permanent testing frame. A 19.0 mm gusset plate with three bolt-holes was attached at the top of the braces. All the gusset plates were detailed such that a straight-line plastic hinge could form perpendicular to the brace axis. The top brace gusset plate fit between two beam connection plates with vertically slotted holes. Three bolts connecting the brace gusset plate and the outer beam connection plates were fastened "finger tight" to transfer lateral loads, while permitting vertical movement along the slotted holes. The floor beam above the braces consisted of a Class 1 wide-flange member (W200x86) made of G40.21-350W steel, which was connected to two pin-ended column members below.

Two prototype specimens were fabricated by Solid Rock Steel Fabricating Company of Surrey, BC, using common fabrication techniques and tolerances. (Complete fabrication drawings can be found in Appendix B.) The first specimen was tested with the braces left as hollow tubes, while the brace members of the second specimen were filled with concrete.
4.4 Test 1: Hollow Tube (HSS) Braces

4.4.1 Description of Test 1 Set-up

Each specimen was bolted at the brace base plates to the permanent testing frame. The lateral load was applied to the upper floor beam by the hydraulic actuator, as shown in Figure 4.4.1. Lateral support brackets with rollers were mounted on the permanent testing frame to provide stability against out-of-plane movements of the floor beam. Whitewashing the steel surfaces prior to testing facilitated visual observation of the yield zones.
4.4.2 Instrumentation

Considering the small number of samples, extensive instrumentation was utilized to monitor all aspects of the specimens' behaviour, which would help in the interpretation of results and enable reconstruction of events during testing. A schematic denoting the location of instruments on the test specimen is shown in Figure 4.4.2, and a description of all instrumentation follows in Table 4.4.1. (Complete instrumentation details can be found in Appendix A.) A load cell was integrated with the hydraulic actuator (Figure 4.4.3) and both column posts were strain gauged and calibrated to measure axial loads. Displacement transducers (LVDTs and string potentiometers) were placed at various locations to measure horizontal, vertical and differential (between brace gusset plate and
beam connection plate) movements of the test specimen (Figure 4.4.4). A number of strain gauges were mounted on the specimen to collect localized behaviour response data, and also to verify other instrument measurements (Figure 4.4.5). All instrument data channels were connected to a central 40-channel data acquisition system, which was in turn connected to a computer for data collection (Figure 4.4.6). The data was sampled at 10 samples per second.
Table 4.4.1 Test Instrumentation Set-up

<table>
<thead>
<tr>
<th>Channel</th>
<th>Description</th>
<th>Instrumentation</th>
<th>Location on Specimen</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>actuator load</td>
<td>load cell</td>
<td>integral with actuator</td>
</tr>
<tr>
<td>2</td>
<td>actuator stroke</td>
<td>displacement transducer</td>
<td>integral with actuator</td>
</tr>
<tr>
<td>3</td>
<td>frame lateral displacement</td>
<td>string potentiometer</td>
<td>south end of beam</td>
</tr>
<tr>
<td>4</td>
<td>beam vertical displacement</td>
<td>LVDT</td>
<td>top of beam at mid-span</td>
</tr>
<tr>
<td>5</td>
<td>slotted connection movement</td>
<td>LVDT</td>
<td>centreline of braces, attached to beam</td>
</tr>
<tr>
<td>6</td>
<td>south column load</td>
<td>2 double-rosette strain</td>
<td>mid-height south column post, attached on</td>
</tr>
<tr>
<td></td>
<td></td>
<td>strain gauges</td>
<td>opposite sides</td>
</tr>
<tr>
<td>7</td>
<td>north column load</td>
<td>2 double-rosette strain</td>
<td>mid-height north column post, attached on</td>
</tr>
<tr>
<td></td>
<td></td>
<td>strain gauges</td>
<td>opposite sides</td>
</tr>
<tr>
<td>8</td>
<td>south brace, long, strain</td>
<td>strain gauge</td>
<td>mid-length south brace, east face</td>
</tr>
<tr>
<td>9</td>
<td>south brace, long, strain</td>
<td>strain gauge</td>
<td>mid-length south brace, west face</td>
</tr>
<tr>
<td>10</td>
<td>south brace, long, strain</td>
<td>strain gauge</td>
<td>mid-length south brace, south face</td>
</tr>
<tr>
<td>11</td>
<td>south brace, long, strain</td>
<td>strain gauge</td>
<td>south brace, north face mid-length</td>
</tr>
<tr>
<td>12</td>
<td>north brace, long, strain</td>
<td>strain gauge</td>
<td>north brace, east face mid-length</td>
</tr>
<tr>
<td>13</td>
<td>north brace, long, strain</td>
<td>strain gauge</td>
<td>north brace, west face mid-length</td>
</tr>
<tr>
<td>14</td>
<td>north brace, long, strain</td>
<td>strain gauge</td>
<td>north brace, south face mid-length</td>
</tr>
<tr>
<td>15</td>
<td>north brace, long, strain</td>
<td>strain gauge</td>
<td>north brace, north face mid-length</td>
</tr>
<tr>
<td>16</td>
<td>south gusset plate strain</td>
<td>strain gauge</td>
<td>south gusset plate, east face</td>
</tr>
<tr>
<td>17</td>
<td>south gusset plate strain</td>
<td>strain gauge</td>
<td>south gusset plate, west face</td>
</tr>
<tr>
<td>18</td>
<td>north gusset plate strain</td>
<td>strain gauge</td>
<td>north gusset plate, east face</td>
</tr>
<tr>
<td>19</td>
<td>north gusset plate strain</td>
<td>strain gauge</td>
<td>north gusset plate, west face</td>
</tr>
<tr>
<td>20</td>
<td>south top gusset plate, east</td>
<td>triple rosette strain</td>
<td>south top gusset plate, east face</td>
</tr>
<tr>
<td></td>
<td>horizontal strain</td>
<td>gauge</td>
<td>oriented along horizontal axis</td>
</tr>
<tr>
<td>21</td>
<td>south top gusset plate, east</td>
<td>triple rosette strain</td>
<td>south top gusset plate, east face</td>
</tr>
<tr>
<td></td>
<td>vertical strain</td>
<td>gauge</td>
<td>oriented along vertical axis</td>
</tr>
<tr>
<td>22</td>
<td>south top gusset plate, east</td>
<td>triple rosette strain</td>
<td>south top gusset plate, east face at 45</td>
</tr>
<tr>
<td></td>
<td>diagonal (45 degree) strain</td>
<td>gauge</td>
<td>degrees to horizontal axis</td>
</tr>
<tr>
<td>23</td>
<td>south top gusset plate, west</td>
<td>triple rosette strain</td>
<td>south top gusset plate, west face</td>
</tr>
<tr>
<td></td>
<td>horizontal strain</td>
<td>gauge</td>
<td>oriented along horizontal axis</td>
</tr>
<tr>
<td>24</td>
<td>south top gusset plate, west</td>
<td>triple rosette strain</td>
<td>south top gusset plate, west face</td>
</tr>
<tr>
<td></td>
<td>vertical strain</td>
<td>gauge</td>
<td>oriented along vertical axis</td>
</tr>
<tr>
<td>25</td>
<td>south top gusset plate, west</td>
<td>triple rosette strain</td>
<td>south top gusset plate, west face at 45</td>
</tr>
<tr>
<td></td>
<td>diagonal (45 degree) strain</td>
<td>gauge</td>
<td>degrees to horizontal axis</td>
</tr>
<tr>
<td>26</td>
<td>north top gusset plate, east</td>
<td>triple rosette strain</td>
<td>north top gusset plate, east face</td>
</tr>
<tr>
<td></td>
<td>horizontal strain</td>
<td>gauge</td>
<td>oriented along horizontal axis</td>
</tr>
<tr>
<td>27</td>
<td>north top gusset plate, east</td>
<td>triple rosette strain</td>
<td>north top gusset plate, east face</td>
</tr>
<tr>
<td></td>
<td>vertical strain</td>
<td>gauge</td>
<td>oriented along vertical axis</td>
</tr>
<tr>
<td>28</td>
<td>north top gusset plate, east</td>
<td>triple rosette strain</td>
<td>north top gusset plate, east face at 45</td>
</tr>
<tr>
<td></td>
<td>diagonal (45 degree) strain</td>
<td>gauge</td>
<td>degrees to horizontal axis</td>
</tr>
<tr>
<td>29</td>
<td>north top gusset plate, west</td>
<td>triple rosette strain</td>
<td>north top gusset plate, west face</td>
</tr>
<tr>
<td></td>
<td>horizontal strain</td>
<td>gauge</td>
<td>oriented along horizontal axis</td>
</tr>
<tr>
<td>30</td>
<td>north top gusset plate, west</td>
<td>triple rosette strain</td>
<td>north top gusset plate, west face</td>
</tr>
<tr>
<td></td>
<td>vertical strain</td>
<td>gauge</td>
<td>oriented along vertical axis</td>
</tr>
<tr>
<td>31</td>
<td>north top gusset plate, west</td>
<td>triple rosette strain</td>
<td>north top gusset plate, west face at 45</td>
</tr>
<tr>
<td></td>
<td>diagonal (45 degree) strain</td>
<td>gauge</td>
<td>degrees to horizontal axis</td>
</tr>
<tr>
<td>32</td>
<td>top gusset plate lateral displacement</td>
<td>string potentiometer</td>
<td>top gusset plate, west face, just below slotted connector plate</td>
</tr>
</tbody>
</table>
Figure 4.4.3 Actuator Load Cell and Beam LVDT

Figure 4.4.4 LVDT and String Potentiometers Connected to Specimen
Chapter 4

Quasi-Static Cyclic Testing

Figure 4.4.5 Brace Gusset Plate LVDT and Triple Rosette Strain Gauges

Figure 4.4.6 Quasi-Static Cyclic Testing Data Acquisition System
4.4.3 Test Protocol

The testing program chosen for the experiment was quasi-static cyclic loading as per the Applied Technology Council ATC-24 loading protocol [ATC, 1992]. The load history consisted of a series of stepwise increasing deformation cycles, given as multiples of the yield displacement ($\delta_y$), as shown in Figure 4.4.7. The average loading rate varied between 14 and 28 mm/min, in terms of the total displacement in a cycle divided by the total period. This slow, stepwise method of testing is the preferred standard for obtaining information needed to develop design and detailing procedures and results can be considered somewhat conservative [Krawinkler, 1988]. The yield displacement was chosen at the first noticeable deviation from a linear-elastic curve for the first specimen, which was at 28 mm.

![Figure 4.4.7 Nominal Displacement-Controlled Testing Pattern](image)

Due to a sluggish response of the actuator to the control signal, it was found that displacement amplitude peaks were somewhat rounded and the desired displacements were not achieved. It was thus decided to use a sinusoidal input function instead of the linear sawtooth pattern as shown in Figure 4.4.7. As a result, the amplitude change around the peak values was very low, which allowed the actuator displacement to catch...
up with the input function. The consequence of this decision was that the loading rate was not constant throughout a cycle. This compromise was deemed acceptable, as a variation in the loading rate, within acceptable limits, would not significantly influence the results.

The complete sinusoidal loading protocol for Test 1 is shown in Table 4.4.2.

<table>
<thead>
<tr>
<th>Step</th>
<th>Increment</th>
<th>Portion of $\delta_{\text{yield}}$</th>
<th>Amplitude (mm)</th>
<th>Period (sec)</th>
<th>No. of Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$n_0$</td>
<td>0.25</td>
<td>7</td>
<td>120</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>$n_0$</td>
<td>0.50</td>
<td>14</td>
<td>240</td>
<td>3</td>
</tr>
<tr>
<td>3</td>
<td>$n_0$</td>
<td>0.75</td>
<td>21</td>
<td>360</td>
<td>3</td>
</tr>
<tr>
<td>4</td>
<td>$n_1$</td>
<td>1.00</td>
<td>28</td>
<td>480</td>
<td>3</td>
</tr>
<tr>
<td>5</td>
<td>$n_2$</td>
<td>2.00</td>
<td>56</td>
<td>480</td>
<td>3</td>
</tr>
<tr>
<td>6</td>
<td>$n_3$</td>
<td>3.00</td>
<td>84</td>
<td>720</td>
<td>3</td>
</tr>
<tr>
<td>7</td>
<td>$n_4$</td>
<td>4.00</td>
<td>112</td>
<td>960</td>
<td>2</td>
</tr>
<tr>
<td>8</td>
<td>$n_5$</td>
<td>5.00</td>
<td>140</td>
<td>1200</td>
<td>2</td>
</tr>
</tbody>
</table>

**4.4.4 Test Results**

Testing of the first specimen continued for 12 inelastic cycles (21 cycles in total) until a tension failure occurred in one of the braces (i.e. complete fracture across the brace cross-section). The maximum beam displacement was ±140 mm, which amounts to approximately 3.8% storey drift. The peak lateral load applied to the system was 263 kN. The hysteretic behaviour of the hollow tube specimen is shown in Figure 4.4.8. The data channels used to develop this curve were the force measurement from the load cell at the top of the beam (Ch. 1), and the displacement measurement from the string potentiometer at the end of the beam (Ch. 3).
4.5 Test 2: Concrete-Filled Tube (CFT) Braces

4.5.1 Description of Test 2 Set-up

The test set-up and equipment used for Test 2 was the same as for Test 1, with the exception that the specimen now had concrete-filled tubes (CFT). The purpose of the concrete was to study the effects of the fill on local buckling characteristics of the braces. The concrete used to fill the tubes had a 28-day strength of approximately 30 MPa. No effort was made to achieve a particular concrete strength, as it has been found that a variation of the concrete strength in the 28 to 55 MPa range would not have a significant effect on the results [Liu and Goel, 1988].

4.5.2 Instrumentation

The original instrumentation arrangement remained the same as for Test 1, details of which can be found in Section 4.4.2. However, an additional motion capture instrumentation system was added for Test 2. The Visualeyez™ system [PTI, 2001] is an
active-optical real-time motion tracking system that can collect three-dimensional coordinates for instrumented objects in motion. The instrumentation system included one Visualeyez™ tracker, 32 light emitting diode (LED) markers, one 64-channel target control module (TCM) used to activate and control the LED markers, and one wireless transmission system (transmitter and receiver) to send signals between the TCM and tracker. All data collected from the tracker were sent to a computer with the VZSoft™ Graphical User Interface software to view, edit and export the data. The complete motion capture system instrumentation is shown in Figure 4.5.1 and Figure 4.5.2.
4.5.3 Test Protocol

The test protocol was very similar to that used in Test 1. The yield displacement was found to be 30 mm for the second test set-up, and thus the displacement amplitudes of each step were modified by the appropriate amount. The loading rate also changed slightly from the first test, according to the new displacement amplitude and period for each cycle. The complete sinusoidal loading protocol for Test 2 is shown in Table 4.5.1.
Table 4.5.1 Test 2 Loading Protocol

<table>
<thead>
<tr>
<th>Step</th>
<th>Increment</th>
<th>Portion of $\delta_{\text{yield}}$</th>
<th>Amplitude (mm)</th>
<th>Period (sec)</th>
<th>No. of Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$n_0$</td>
<td>0.25</td>
<td>7.5</td>
<td>120</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>$n_0$</td>
<td>0.50</td>
<td>15</td>
<td>240</td>
<td>3</td>
</tr>
<tr>
<td>3</td>
<td>$n_0$</td>
<td>0.75</td>
<td>22.5</td>
<td>360</td>
<td>3</td>
</tr>
<tr>
<td>4</td>
<td>$n_1$</td>
<td>1.00</td>
<td>30</td>
<td>480</td>
<td>3</td>
</tr>
<tr>
<td>5</td>
<td>$n_2$</td>
<td>2.00</td>
<td>60</td>
<td>480</td>
<td>3</td>
</tr>
<tr>
<td>6</td>
<td>$n_3$</td>
<td>3.00</td>
<td>90</td>
<td>720</td>
<td>3</td>
</tr>
<tr>
<td>7</td>
<td>$n_4$</td>
<td>4.00</td>
<td>120</td>
<td>960</td>
<td>2</td>
</tr>
<tr>
<td>8</td>
<td>$n_5$</td>
<td>5.00</td>
<td>150</td>
<td>1200</td>
<td>2</td>
</tr>
</tbody>
</table>

4.5.4 Test Results

The CFT brace specimen was still intact after completion of the proposed testing protocol. It was decided to continue cycling the specimen at the maximum displacement amplitude ($\pm 150$ mm) until fracture and subsequent failure. This displacement was governed by limitations of the testing apparatus.

Testing of the second specimen continued for 34 inelastic cycles (43 cycles in total) until both braces had no further tension capacity (i.e. both braces were torn across the entire cross-section). The maximum beam displacement was $\pm 150$ mm, which amounts to approximately 4.1% storey drift. The peak lateral load applied to the system was 322 kN. The hysteretic behaviour of the concrete-filled tube specimen is shown in Figure 4.5.3 and the response to failure is shown in Figure 4.5.4. The data channels used to develop these curve were the force measurement from the load cell at the top of the beam (Ch. 1), and the displacement measurement from the string potentiometer at the end of the beam (Ch. 3).
Figure 4.5.3 Load-Displacement Curve for Test 2 (Step 1 – Step 8, Cycle 2)

Figure 4.5.4 Load-Displacement Curve for Test 2 to Failure (Step 8, Cycle 3 - 23)
5.1 Introduction

A number of important findings were made through observations during testing and by the comparison of results from Tests 1 and 2. A discussion of the outcomes of the experimental testing is presented in this section, along with the significance of these results. A collection of photographs taken during each test can be found in Appendix C.

5.2 Comparison to Numerical Prediction

A calculation was done before testing to predict the response of the braced frame in the linear-elastic range until brace compression buckling would occur. (Details of this calculation are provided on a Mathcad [MathSoft, Inc., 2001] sheet included in Appendix D.) The predicted load-displacement relationship for the hollow tube braces is shown by the dashed line in Figure 5.2.1. The critical buckling load, $P_e$, was calculated using the Euler Formula (5.1), where $E$ is the elastic modulus, $I$ is the moment of inertia of the section, $k$ is the effective length factor and $L$ is the length of the brace. The critical buckling load was estimated to be 326 kN, assuming an effective length factor of 0.9 according to brace and gusset plate geometry and an anticipated buckling shape. This corresponded to a total lateral load of 259 kN at brace buckling.

$$P_e = \frac{\pi^2 EI}{(kL)^2} \quad (5.1)$$

As shown in Figure 5.2.1, there is a large discrepancy between the frame displacement in the numerical prediction and in the test results at the instant that buckling occurs, although the buckling load is quite similar. Similar inconsistencies were seen between the analytical prediction of yield displacement and the observed experimental values in
testing done by others with bolted X-plate ADAS devices between chevron braces and the supporting beam [Whittaker et al., 1989], as described in Section 2.2.2.

![Graph showing comparison of experimental results with numerical prediction]

Figure 5.2.1 Comparison of Predicted and Experimental Results in Linear Range

Due to these inconsistencies, it became important to re-examine the assumptions made in the prediction to rationalize the differences between expected and observed results. In the hand calculation, the displacement in the model was applied directly to the top of the braces, as the slotted bolted connection was assumed to offer full shear and displacement transfer from the beam to the braces. In contrast, the true experimental specimen had the point of load application just above the beam, and transferred all forces and motions to the braces through the slotted bolted connection.

Upon closer examination of the experimental data, evidence of a discrepancy between the displacement at the beam and the displacement at the top of the braces was observed. The differences were seen between the measured translations with the string potentiometer connected to the beam and the string potentiometer connected to the top of the braces. An example of this data is shown for the first non-linear step in the displacement-controlled cycles for Test 1 on the hollow tube brace specimen (Figure 5.2.2).
Chapter 5 Discussion of Experimental Results

Figure 5.2.2 Load-Displacement Curve for Test 1 (Step 5)

The data collected by the Visualeyez™ system during Test 2 also indicated significant differences in lateral motions at the beam level and at the top of the braces. For example, lateral displacements measured at LED markers 13 and 23 (Figure 5.2.3) are shown in Figure 5.2.4, Figure 5.2.5 and Figure 5.2.6 for various load reversal cycles during Test 2.

Figure 5.2.3 Arrangement of Selected LED Markers in Test 2
Figure 5.2.4 Displacement Curve for Test 2 (Step 3, Cycle 1)

Figure 5.2.5 Displacement Curve for Test 2 (Step 4, Cycle 1)

Figure 5.2.6 Displacement Curve for Test 2 (Step 5, Cycle 1)
From the Visualeyez™ data, it appears that, until the onset of brace buckling, there is a linearly varying discrepancy between the beam and brace displacement, with the top of the braces moving at about 70% of the beam motion. Immediately following buckling, the top of the braces begin to translate with the beam, and the deficiency in brace displacement shows only a slight increase during the remaining post-buckling cycles. The average residual offset between the beam and the top of the braces appears to be approximately 7 mm to 10 mm from the end of Step 5 (2 $\delta_y$) to the end of Step 8 (5 $\delta_y$).

In addition, the three-dimensional coordinates captured with the Visualeyez™ system gave further insight into the potential sources for differences between predicted and observed results. At time instances of peak lateral deformation, the displacements measured at LED markers 13, 15, 23 and 25 (Figure 5.2.3) are shown in Table 5.2.1, Table 5.2.2 and Table 5.2.3. Recall that the direction of motion was measured as vertical along the x-axis, lateral along the y-axis, and out-of-plane along the z-axis.

<table>
<thead>
<tr>
<th>LED Marker</th>
<th>Maximum Displacement (mm)</th>
<th>Minimum Displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\Delta X$</td>
<td>$\Delta Y$</td>
</tr>
<tr>
<td>13</td>
<td>1.8</td>
<td>17.3</td>
</tr>
<tr>
<td>15</td>
<td>5.7</td>
<td>17.0</td>
</tr>
<tr>
<td>23</td>
<td>3.8</td>
<td>10.8</td>
</tr>
<tr>
<td>25</td>
<td>3.6</td>
<td>10.8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LED Marker</th>
<th>Maximum Displacement (mm)</th>
<th>Minimum Displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\Delta X$</td>
<td>$\Delta Y$</td>
</tr>
<tr>
<td>13</td>
<td>-3.7</td>
<td>52.8</td>
</tr>
<tr>
<td>15</td>
<td>-3.7</td>
<td>52.6</td>
</tr>
<tr>
<td>23</td>
<td>-9.8</td>
<td>47.5</td>
</tr>
<tr>
<td>25</td>
<td>-4.7</td>
<td>47.4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LED Marker</th>
<th>Maximum Displacement (mm)</th>
<th>Minimum Displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\Delta X$</td>
<td>$\Delta Y$</td>
</tr>
<tr>
<td>13</td>
<td>-15.3</td>
<td>147.8</td>
</tr>
<tr>
<td>15</td>
<td>-15.6</td>
<td>147.6</td>
</tr>
<tr>
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<td>-33.1</td>
<td>139.5</td>
</tr>
<tr>
<td>25</td>
<td>-10.9</td>
<td>140.8</td>
</tr>
</tbody>
</table>
Chapter 5  

Discussion of Experimental Results

The x- and z-axis data shown in the previous tables indicate that there was significant out-of-plane motion and rotation in the specimen at the beam and slotted connection. Thus, data measured by the string potentiometers may have included these additional motions instead of recording pure lateral deformation. A comparison between beam and brace motions measured by the string potentiometers and the Visualeyez\textsuperscript{TM} system is shown in Table 5.2.4.

<table>
<thead>
<tr>
<th>Test 2</th>
<th>Maximum Displacement (mm)</th>
<th>Minimum Displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>String Pot.</td>
<td>Visualeyez\textsuperscript{TM}</td>
</tr>
<tr>
<td>4</td>
<td>Beam</td>
<td>28.8</td>
</tr>
<tr>
<td></td>
<td>Braces</td>
<td>20.6</td>
</tr>
<tr>
<td>5</td>
<td>Beam</td>
<td>56.8</td>
</tr>
<tr>
<td></td>
<td>Braces</td>
<td>54.0</td>
</tr>
<tr>
<td>8</td>
<td>Beam</td>
<td>144.8</td>
</tr>
<tr>
<td></td>
<td>Braces</td>
<td>139.0</td>
</tr>
</tbody>
</table>

It appears that the inconsistencies between the string potentiometer and Visualeyez\textsuperscript{TM} data seem to diminish as the testing continues, which may be related to the out-of-plane deformations of the specimen. In the cycles when buckling first occurred, the specimen was twisting and rotating, and plastic hinges were just starting to form as out-of-plane deformations began. In the post-buckling cycles, the hinge locations were better defined and less out-of-plane motion was detected at the beam level.

In general, the specimen displacement patterns observed with the Visualeyez\textsuperscript{TM} motion capture system were similar in magnitude and form to those found with the string potentiometers and other specimen instrumentation. The Visualeyez\textsuperscript{TM} data does illustrate that the pure lateral motions of the test specimen were closer to the numerically predicted values than first indicated by the string potentiometer data. However, because the motion capture system was not used during Test 1, it will be used only as a means for validation of the remaining experimental data.
5.3 Slotted Connection Performance

One concern that was raised before testing began was the reliability of the vertical slotted connection (VSC), namely the potential for binding of the bolts along the slotted-holes. The probable distortion of the top brace gusset plate, due to plastic hinge formation after brace buckling, was expected to affect the alignment of the brace plate and beam connection plate. To address this concern, the top brace gusset plate was designed having a greater thickness than the bottom gusset plate in an attempt to minimize the effects of deformation during plastic hinge formation.

During testing, plastic deformations of the brace plates occurred directly beyond the ends of the braces at the reduced plate section (Figure 5.3.1 and Figure 5.3.2). The VSC performed very well, in terms of allowing specimen vertical movement, and bolt binding was not an issue (Figure 5.3.3). The length of the slots was just sufficient to permit the travel of the bolts during the cyclic testing. At the extreme displacements during the second test, the bolts initiated bearing at the ends of the slots, which is reflected in the slight increase in load during large displacement cycles (Figure 4.5.3).

![Figure 5.3.1 Plastic Hinge Formation along Reduced Plate Section](image)
Chapter 5  Discussion of Experimental Results

Figure 5.3.2 Plastic Hinge Formation at Bottom Gusset Plate

Figure 5.3.3 Bolt Movement along Vertical Slotted Connection

The VSC also affected the lateral stiffness of the braced frame system. As described above, there were larger displacements than expected for given lateral loads during the experimental testing. It appears that the small gaps between the bolts and slots in the VSC
detail permitted horizontal slip as well as rotation of the top brace gusset plate in the three slotted holes (Figure 5.3.4). This slip between bolts and plates would result in connections that could be idealized as translational or rotational springs rather than rigid restraints. In the out-of-plane direction, the gaps between the chevron top brace gusset plate and the slotted beam connection plate may have permitted some unexpected translation and rotation as well.

5.4 Chevron Brace Performance

5.4.1 Global Behaviour

As expected, the chevron braces experienced out-of-plane buckling at loads predicted using column buckling theory (Figure 5.4.1). For the hollow tube brace specimen, the overall profile of the buckled compression brace had a distinct bilinear shape, due to the concentration of a plastic hinge directly at the mid-length of the brace (Figure 5.4.2). The concrete-filled tube brace, in contrast, had a much smoother curvilinear buckled shape (Figure 5.4.3). In the latter case, the local buckles were distributed along a greater length of the member, avoiding areas of high strain, and thus maintaining the integrity of the concrete-filled tube for a greater number of cycles.
Chapter 5  

Discussion of Experimental Results

Figure 5.4.1 Global Behaviour of Chevron Braced Frame

Figure 5.4.2 Global Buckling Shape of Hollow Tube Brace
5.4.2 Local Buckling Effects

The key to the superior performance of the Test 2 specimen was the role of the concrete fill in reducing severe strains caused by local buckling in the tube walls. In testing done by others [Goel and Liu, 1987], it was found that hollow tubes exhibit early fractures in regions of plastic hinge formation due to local buckling effects, while concrete fill works compositely with the steel tubes, resulting in more ductile behaviour and better energy dissipation characteristics. These findings were reflected in the results of the VSC chevron brace testing. The local buckling was concentrated in a central location in the hollow tube braces, which can be seen quite clearly by the pronounced indentation on one face of the tube wall, while adjacent faces buckled outward (Figure 5.4.4). On the other hand, the local buckling pattern observed on the concrete-filled tube braces stretched over a longer region along the mid-length of the tubes, and only consisted of outward buckles, since inward buckling was resisted by the concrete fill (Figure 5.4.5).
Chapter 5

Discussion of Experimental Results

Figure 5.4.4 Local Buckling Pattern for Hollow Tube Brace

Figure 5.4.5 Local Buckling Pattern for Concrete-Filled Tube Brace
5.4.3 Fracture and Failure Characteristics

The fracture pattern for each of the two steel tube systems was quite distinct. For the hollow tube braces, cracks were concentrated at the corners of the tube walls at locations of high residual stresses and intense strain hardening (Figure 5.4.6). The concrete-filled tube braces maintained their integrity at the corners for a much larger number of loading cycles, and when fracture did occur, the cracks formed across the entire face of the tube wall (Figure 5.4.7). Thus, the controlling of local buckling effects in the tube walls clearly determined the progressive state of failure for the two brace specimens.

Figure 5.4.6 Crack Formation for Hollow Tube Brace
5.5 Ductility

The overall ductility of the braced frame was low, in terms of the structure's ability to withstand considerable deformations without a substantial loss of strength. This was not unexpected, however, as bracing systems with compression buckling elements typically suffer a significant drop in strength and stiffness following the peak storey shear. However, the test specimen did continue to carry a portion of its initial load capacity for a significant number of inelastic load reversal cycles before failure, indicating that there was some ductility in the brace elements.

5.5.1 Strength Degradation

The reduction in strength of the specimen occurred immediately following buckling of the compression brace in the system. The storey shear carried by the frame at twice the yield displacement (2 \( \delta_y \)) was reduced to about 55% of the peak lateral capacity of the
system at first yielding ($\delta_y$). At three times the yield displacement ($3\delta_y$), the storey shear was approximately 35% of the peak lateral capacity. For the hollow tube brace specimen, at the CSA code 2% storey drift limit, the frame capacity was about 95 kN at the first cycle of loading and 80 kN for the two subsequent displacement cycles. For the concrete-filled tube specimen, at 2% storey drift, the frame capacity was about 130 kN at the first cycle of loading and 110 kN for the two subsequent displacement cycles.

### 5.5.2 Stiffness Degradation

The stiffness of the specimen changed significantly after buckling of each of the brace members, which lead to the somewhat pinched hysteresis loop that is characteristic of tension-compression bracing systems. The stiffness of the frame was reduced to about 40% of its initial value in the second inelastic cycle (following buckling), and the load-displacement curve exhibited a long yield plateau region for deformations beyond the yield displacement ($\delta_y$) at later stages of testing. This implies that if an earthquake pulse large enough to induce brace buckling occurs early in an event, the residual stiffness of the frame may not be sufficient to keep the displacements of the structure within code drift limitations for the entire duration of the shaking.

### 5.6 Energy Dissipation Capacity

Although the ductility of the system (as defined in Section 5.5) was low, the specimens endured a large number of inelastic load reversal cycles before fracture and ultimate failure. Therefore, the energy dissipation capacity came from the specimen’s ability to undergo many displacement cycles during testing. The concrete-filled tube braces performed very well because, as local buckling effects were minimized, the number of cycles before fracture was extended, as was the service life of the brace. Due to displacement limitations of the testing apparatus, it was not possible to increase the displacement amplitude of the cycles. It was thus decided to continue cycling at ±150 mm ($5\delta_y$) until fracture of the braces occurred.
Chapter 5 Discussion of Experimental Results

The hysteresis loops did not show much degradation when displacements remained constant for a number of cycles. This stable response for repeated cycles demonstrates the frame’s capacity to dissipate energy without rapid deterioration of the ductile brace fuse element.

The energy dissipated in a cycle of testing is equal to the area under the load-displacement curve for that cycle. The shaded areas in Figure 5.6.1 give a sample of the contributing regions under the hysteresis curve for a few inelastic cycles during Test 1. For the hollow tube brace specimen, the total energy dissipated was equal to 106 kJ. The amount of energy dissipated by the concrete-filled tube brace specimen was much larger than the hollow tube brace specimen, and totalled 626 kJ.

Figure 5.6.1 Illustration of Energy Dissipation by Area Under Hysteresis Curve
CHAPTER 6
ANALYTICAL MODELLING

6.1 Introduction

The analytical phase of the research aimed to replicate the behaviour observed in the experimental tests, using a computer model of the VSC chevron braced frame. The intent of the analytical study was to build a detailed model of the proposed system, calibrated to the actual measured response of the test specimen, before including the VSC module into a larger building model undergoing a pushover or non-linear time-history analysis. A simplified model was also developed, using different software and modelling techniques, to explore a less detailed approach to represent the non-linear brace behaviour.

By comparing both models with the observed experimental results, an assessment could be made as to the suitability of various commercially available software programs to model the complex non-linear behaviour of the VSC chevron brace system.

6.2 Detailed Model

Past studies of various available analytical brace models have indicated the importance of accurate hysteretic modelling. Such a model must capture the influence of phenomena affecting brace non-linear behaviour, including: member buckling, material yielding, local buckling and material strain hardening [Remennikov and Walpole, 1997]. For this reason, a model consisting of linear elements and springs was deemed most appropriate for the detailed modelling of the VSC chevron brace system. This approach was chosen over a finite element model (FEM) because of the extensive computational time that would have been required to accurately model the connection details. It was decided not to use a phenomenological model either, because of the heavy reliance on case-specific experimental data that would have been required to create such a model.
Chapter 6

Analytical Modelling

An important selection that needed to be made before modelling began was the computer software that would be used to create the detailed model of the VSC chevron brace system. It was found that RUAUMOKO [Carr, 2001] was a suitable program for the analytical study, based on the non-linear pushover and time-history analysis capabilities, the option of a three-dimensional modelling environment and the vast library of hysteresis rules available. The program CANNY99 [Li, 2000] was considered for the analysis but deemed less desirable because it lacked a steel hysteresis model for buckling braces. ETABS [CSI, 2002] was also considered for the task, but was judged less suitable for similar reasons.

6.2.1 RUAUMOKO Computer Program

The program RUAUMOKO was developed by Dr. Athol Carr at the University of Canterbury in New Zealand in 1996. The most current release of the program includes a three-dimensional version, RUAUMOKO-3D, which has capabilities to perform a non-linear dynamic analysis of general framed structures in response to ground accelerations, time varying force excitations or displacement time-histories. There are also provisions for an adaptive pushover analysis for either monotonic or cyclic loading.

There are several options available in the program for the modelling of mass, damping and stiffness parameters, and a variety of members are available to represent the main structural system and its supporting or adjoining elements. The program also includes a vast library of over forty different hysteresis rules that may be used to describe the force-displacement relationship in the different frame, spring, contact and foundations elements in the model.

The program can be run in an interactive mode with an input data file in the form of an ASCII text file. After the data has been read and checked with the pre-processor, the analysis begins and on-screen graphics are available for viewing the frame geometry and input excitations. Next, the mode shape results are available, followed by the deformed shape results during the time-history analysis. An output file is produced, capturing the
results of the various analyses performed, which can be viewed in a text editor program. A post-processor program, DYNAPLOT or DYNAPLOT-3D, can also be used to graphically display the results contained in the output file in the form of various time-history plots or hysteresis loops for selected members or nodes.

It was decided that a two-dimensional model would be constructed in RUAUMOKO to replicate the VSC chevron braced frame behaviour. This could be expanded in further studies to a more extensive three-dimensional model, once the former has been calibrated to test results.

### 6.2.2 Description of Model

The geometry for the VSC chevron braced frame model was taken from the experimental test specimens. All nodes were positioned along the centrelines of the members at key locations or intersections of elements. Figure 6.2.1 shows an image of the frame geometry as displayed in RUAUMOKO.

The one-storey, one-bay structure was modelled using frame elements for all members, including a beam member, pin-ended column members and brace members with rotational fixity at the ends. The slotted connection between the braces and the beam was modelled by slaving the horizontal displacements of the node at the top of the braces and the node at the mid-span of the beam, while removing the vertical restraint from each of these nodes, leaving the node at the top of the braces free to translate along the y-direction independently from the beam.

The time-varying displacement excitation was applied along the x-direction at the node located at the mid-span of the beam. All displacement increments were identical to those used in the loading protocol for testing.
6.2.3 Brace Hysteresis Properties

The non-linearities in geometry and material behaviour observed for the chevron braces during testing were critical elements to capture in the analytical model of the system. An appropriate model was thus needed to describe the axial force-displacement relationship of the braces throughout their full displacement range. The hysteresis rule selected for these frame elements was the Remennikov Steel Brace Member Hysteresis model (Figure 6.2.2), taken from the library of hysteresis models in RUAUMOKO. The model is designed to represent the out-of-plane buckling of a steel brace member and requires input data for the brace moment of inertia, section modulus, effective length factor, strain hardening factors and tangent moduli corresponding to varying levels of axial force.

Refining of the brace hysteresis rule was an important stage in the model formation, and was done in collaboration with Dr. Alexander Remennikov, the developer of the brace buckling hysteresis routine in RUAUMOKO.
6.2.4 Analysis Results

Preliminary results from the model showed that brace yielding, in the form of inelastic buckling, had occurred in both braces at various times during the analysis. The base shear-lateral frame displacement relationship is shown in Figure 6.2.3. The maximum base shear for the braced frame was 245 kN, corresponding to a lateral frame displacement of 8.7 mm. The displacement at the top of the braces was equal to the frame displacement at the top of the beam for the duration of the analysis, due to the node slaving in the model (Section 6.2.2). The general shape of the frame hysteresis loops showed a significant drop in base shear following the peak load, for both the positive and negative displacement cycles, which is characteristic of tension-compression brace systems that undergo severe strength degradation following the buckling of braces.

Figure 6.2.2 RUAUMOKO Hysteresis Rule for Buckling Steel Braces

[Carr, 2001]
6.3 Calibration of Model

For the model calibration, initial results were reviewed to identify differences between the current working model and the baseline data from experimental testing. As a first check, hand calculations were used to confirm the reasonableness of the analytical results, using values determined from first-principles and rational methods to estimate the load-displacement relationship for the frame. This included the linear-elastic behaviour and the elastic buckling load. Following this initial inspection, a brief parametric study and sensitivity analysis were performed to pinpoint key parameters influencing model output. Following these evaluation techniques, the model was adjusted to reflect necessary changes for calibration.

6.3.1 Comparison to Numerical Prediction

Just as a numerical prediction had been used to assess the reasonableness of experimental results, the same estimated values were compared to the initial analytical data. (See Appendix D for details of the numerical prediction.) In terms of a load-displacement
relationship for the frame in the linear-elastic range, the analytical results matched the predicted values extremely well, as shown in Figure 6.3.1. This agreement in results helped to confirm observations made following experimental testing, that the “idealized” representation of the VSC system in the model was responding differently in the linear-elastic range than the actual test specimens. The question of the accuracy of the slotted connection modelling would be examined further in later stages of model calibration. The critical buckling load, as predicted numerically, was in good agreement with the initial analytical results.

![Figure 6.3.1 Comparison of Predicted and Analytical Results in Linear Range](image)

### 6.3.2 Parametric Study and Sensitivity Analysis

The subsequent phase of model calibration entailed taking a rational approach to identify model parameters with variability or uncertainty, and assessing the sensitivity of the model to variations in their values. Model parameters were adjusted up to ±60% of their baseline values to observe changes in model output. The parameters which were varied in this analysis include: brace effective length factor, $k$, brace cross-sectional area, $A$, steel modulus of elasticity, $E$, and effective tangent moduli. The largest discrepancy between
the experimental results and initial analytical results was clearly the initial stiffness of the frame; therefore, the focus of the sensitivity analysis was the response of the frame in the linear-elastic range. Selected results of this study are shown in Figure 6.3.2, keeping in mind that the variations in model parameters shown may be beyond a reasonable range merely to illustrate model sensitivity. (It should be noted that an upper limit was set for the brace compressive strength in the sensitivity analysis, as the intent of the study was to monitor changes in the stiffness of the frame.)

![Figure 6.3.2 Sensitivity Analysis of Model Parameters in Linear Range](image)

As shown by the sensitivity analysis results, even extreme changes in model parameters did not bring the results to within a reasonable range of the observed experimental data. This was also a strong indicator that the discrepancy between the idealized model and the actual test specimen lay in the representation of the overall system, particularly the slotted bolted connection detail, not in the brace member properties or hysteresis rule parameters. The main disparity in results remained, as the experiment showed much larger displacements for given lateral load values than the hand calculation or computer analyses.
6.3.3 Adjustments to Model

With the feedback from the numerical prediction comparison and sensitivity analysis, the model was adjusted to better represent the interaction between the slotted bolted connection and the chevron braces. The simplifying assumptions made during model adjustment were aimed to bring the analytical representation of the frame response closer to the observed behaviour in testing, without changing to a much more complex finite element analysis to represent the VSC detail and non-linear brace elements. It was important, however, to rationalize each adjustment to the model and clearly identify the physical meaning of each additional element, so that the designer retains control over the response of the structure.

6.3.3.1 Serial Spring Model

One explanation for the inconsistencies in experimental and initial analytical results centres around the behaviour of the slotted bolted connection between the beam and the chevron braces. It is likely that there was not an equivalent transfer of translational motion from the actuator and beam to the braces, which would begin to explain why a direct comparison of the beam displacement with the brace displacement at the buckling load would yield such different results. Evidence of this type of discrepancy between the displacement at the beam and the displacement at the top of the braces was present during experimental testing. The evidence was quantified by differences in translations measured with the string potentiometers on the specimen (Figure 5.2.2), as well as differences in translations measured with the Visualeyze™ motion capture system (Figure 5.2.6).

A model adjustment that would correct for the observed discrepancies in beam and brace displacements involved the inclusion of a bilinear spring element connected to the beam in series with the braces. Schematically, the additional element is shown in Figure 6.3.3, with an idealized equivalent spring model to follow (Figure 6.3.4).
Mathematically, the significance of the serial spring is shown in the equations that follow. The difference in beam lateral displacement, $\Delta$, and brace displacement, $\delta_b$, is accounted for in the serial spring displacement, $\delta_s$ (6.1). Rearranging (6.1), an expression for actuator displacement is formed (6.2). By dividing both sides of (6.2) by the applied force, $F$ (6.3), the expression can be rewritten in terms of the frame lateral stiffness, $K$, the serial spring stiffness, $k_s$, and the brace stiffness, $k_b$ (6.4).
\[ \delta_s = \Delta - \delta_b \]  
(6.1)

\[ \Delta = \delta_s + \delta_b \]  
(6.2)

\[ \frac{\Delta}{F} = \frac{\delta_s}{F} + \frac{\delta_b}{F} \]  
(6.3)

\[ \frac{1}{K} = \frac{1}{k_s} + \frac{1}{k_b} \]  
(6.4)

The initial stiffness of the spring was 18000 kN/m, while the secondary stiffness was set to a very high value to simulate a rigid link. The results of the Serial Spring Model analysis demonstrate the effectiveness of the additional element. The load-displacement response in the linear-elastic range captured at the actuator location corresponds much better with the experimental data (Figure 6.3.5), while the load-displacement response captured at the top of the braces agrees with the hand calculation for chevron brace linear-elastic response (Figure 6.3.6).

![Figure 6.3.5 Load-Displacement Curve (at Beam) for Serial Spring Model](image)
The improved correspondence between the experimental and analytical results using the Serial Spring Model further supported the theory that the interaction between the beam and brace members through the VSC element was more complicated than originally assumed. In analogous types of engineering problems, such as soil-structure interaction scenarios, when approaching differences between the seismic excitation at the earthquake source and the excitation at the base of the structure, the two entities may be considered separately to discern the actual input excitation at the point of loading on a structure for the purpose of a building analysis. In a similar fashion, these adjusted model results indicated that the applied displacement-history to the beam through the actuator may not have been the same excitation applied to the top of the braces. As such, the interaction between the VSC element and brace system must be taken into account.

6.3.3.2 Reduced Displacement Amplitude Model

In an attempt to study only the response of the chevron braces, the analytical model was adjusted once more to apply an excitation directly to the top of the braces with a reduced amplitude displacement-history. The new displacement step amplitudes were created
based on the yield displacement value calculated in the numerical prediction. The reduced yield displacement was found to be approximately 9 mm, and the complete displacement-history can be found in Table 6.3.1.

Table 6.3.1 Displacement-History for Reduced Displacement Amplitude Model

<table>
<thead>
<tr>
<th>Step</th>
<th>Increment</th>
<th>Portion of δ\text{yield}</th>
<th>Amplitude (mm)</th>
<th>No. of Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>n₀</td>
<td>0.33</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>n₁</td>
<td>0.67</td>
<td>6</td>
<td>3</td>
</tr>
<tr>
<td>3</td>
<td>n₂</td>
<td>1.00</td>
<td>9</td>
<td>3</td>
</tr>
<tr>
<td>4</td>
<td>n₃</td>
<td>2.00</td>
<td>18</td>
<td>3</td>
</tr>
<tr>
<td>5</td>
<td>n₄</td>
<td>3.00</td>
<td>27</td>
<td>3</td>
</tr>
<tr>
<td>6</td>
<td>n₅</td>
<td>4.00</td>
<td>36</td>
<td>3</td>
</tr>
<tr>
<td>7</td>
<td>n₀</td>
<td>5.00</td>
<td>45</td>
<td>2</td>
</tr>
</tbody>
</table>

Results for the brace response in the Reduced Displacement Amplitude Model analysis are shown in Figure 6.3.7, and can be seen to be in good agreement with the numerical prediction in the linear-elastic range. However, the strength degradation of the frame in the model is much more severe than that observed in testing.

Figure 6.3.7 Load-Displacement Curve for Reduced Displacement Amplitude Model
It should be noted that the overall shape of the load-displacement curve is similar to that obtained in experimental testing, which indicates that RUAUMOKO has the ability to capture, in a general sense, the characteristics of brace buckling behaviour in a tension-compression system under cyclic load reversals. However, the amount of residual strength and level of energy dissipation seen in the RUAUMOKO analysis results remain much lower than in the experimental hysteretic curves. One reason for these discrepancies may be that the brace bending and local buckling of tube walls that occur at plastic hinge locations, where a substantial part of the energy dissipation is taking place, are not accounted for fully in the model. A more sophisticated finite element analysis program may be needed to create an accurate representation of the VSC element and hollow tube braces.

6.4 Simplified Model

Another VSC chevron braced frame model was constructed, this time using SAP2000 NonLinear software [CSI, 2003]. This particular program was chosen for modelling because of its widespread commercial use to model buildings and various other structures for pushover and time-history analyses. The use of SAP2000 required a simplification of the VSC system model, as the program does not have the capability to model complex brace buckling hysteretic behaviour. Instead, plastic hinge locations were specified at the mid-span and the ends of the braces to represent the brace failure mechanism as observed in testing. The simplified model was then subjected to a pushover analysis and the results used for comparison with the experimentally obtained backbone curve.

6.4.1 SAP2000 Computer Program

The SAP2000 computer program was developed by Computers and Structures Inc., with earlier versions of the structural analysis program (SAP) software dating back to the early 1970s at the University of California at Berkeley. The SAP2000 program is currently a widely used structural analysis tool in North America and other parts of the world. The
software has a three-dimensional graphical modelling environment, non-linear capabilities and offers a variety of static and dynamic analysis options.

Included in the program are a number of elements for modelling structures, including: frame, shell, plane, solid, axisymmetric solid and non-linear link elements. The non-linear link element is versatile in itself, as it can be used to represent spring, damper, gap, hook or isolator behaviour in the non-linear range.

### 6.4.2 Description of Model

The frame geometry was similar to the experimental set-up, with the exception that the elevation of the beam was lowered to the height of the top of the concentric braces (Figure 6.4.1). All nodes between frame elements were assigned the appropriate connectivity to adjoining members, and all joints were restricted to in-plane motion, except those located at the mid-span of the braces. To help initiate the out-of-plane buckling action observed in the testing, an initial offset of 8 mm (in the y-direction) was assigned to the mid-span node of the compression brace.

![Figure 6.4.1 SAP2000 Model Frame Geometry](image)
The simplified model also included a spring element connected to the beam in series with the braces, in an attempt to correct for the discrepancy between the beam and brace displacements observed in testing. The spring stiffness was the same value used in the RUAUMOKO model (Section 6.3.3.1).

### 6.4.3 Plastic Hinge Properties

To allow for the deformation and yielding of the braces that would mimic the experimental behaviour as closely as possible, plastic hinge locations were specified at the ends of the braces and at mid-span where plastic regions were observed during testing. The moment-curvature relationship for the hinges was controlled to achieve a load-displacement pushover curve that matched experimental results as closely as possible. The hinge properties specified in the model attempted to accurately represent the yield moments and rotations observed in testing, due to factors such as member overstrength, strain hardening and flexural ductility in the braces (Figure 6.4.2).

![Figure 6.4.2 SAP2000 Plastic Hinge Properties](image-url)
6.4.4 Analysis Results

The results of the pushover analysis show a load-displacement curve typical for a tension-compression bracing system with compression buckling members, and demonstrate good agreement between the experimental and analytical results (Figure 6.4.3). A peak load of 272 kN occurred at a displacement of 30 mm, followed by a significant loss of strength for continued lateral deformation. The load capacity dropped to approximately 32 kN for displacements beyond 75 mm and then remained at a level similar to the experimental results until the end of the displacement cycles.

![Load-Displacement Pushover Curve](image)

Figure 6.4.3 Load-Displacement Pushover Curve for Simplified Model

Using a simplified modelling approach, it was possible to achieve a good agreement between experimental results and SAP2000 pushover analysis results. This demonstrates the suitability of this simplified modelling technique to replicate behaviour observed with a one-bay, one-storey test specimen. This modelling approach will be examined further, to assess its ability to represent a more realistic structure, in the illustrative example that will follow (Chapter 7).
CHAPTER 7

ILLUSTRATIVE EXAMPLE

7.1 Introduction

An illustrative example was created to view the effectiveness of the simplified model in a more realistic pushover analysis using SAP2000. The purpose of the exercise was to incorporate the one-storey, one-bay simplified model into a medium-rise building model, and compare the response of a conventional chevron braced frame system with the VSC chevron braced frame system. A seismic design example of a seven-storey steel building, prepared by The Steel Committee of California [Becker et al., 1988], served as the base building model for this study.

7.2 Description of Building

The building in this example is a seven-storey steel framed structure, designed according to the 1988 edition of the Uniform Building Code [UBC, 1988]. It has chevron bracing in the North-South direction, along column lines 1 and 2, and ductile moment frames in the East-West direction, along column lines A and D (Figure 7.2.1 and Figure 7.2.2). The building is located in Seismic Zone 4, according to the UBC, and the ground below is a dense soil with a depth exceeding 60 m (200 ft). The construction material is 250 MPa (A36) steel and the floors and roof consist of 75 mm (3 in) metal deck with 83 mm (3¼ in) lightweight concrete fill. All of the column, girder and brace members are designed as wide-flange sections, with the brace sizes ranging from W360x64 (W14x43) at the ground floor to W360x101 (W14x68) in the top floor.
Chapter 7

Illustrative Example

Figure 7.2.1 Typical Floor Framing Plan

Figure 7.2.2 Braced Frame Elevation A-A
7.3 Description of Model

It was decided that a pushover analysis would be conducted on the two-dimensional braced frame shown in Figure 7.2.2. The gravity frame was modelled with complete moment connections at all column splices and fixed supports at all column footings. All beams had pinned connections to the columns, as no ductile moment frames were located in the frame elevation being modelled. Braced bays of the modified VSC Model were modelled in a similar fashion to the simplified model described in Section 6.4.2, with a bilinear spring element included in series between the beam and the top of the braces. From the results of the experimental testing, as described in Section 5.2, it was determined that an approximate 7 mm to 10 mm residual offset could be expected between the displacement at the beam and the displacement at the top of the braces using the VSC system; therefore, the bilinear spring followed the model shown in Figure 7.3.1. Plastic hinge locations, in both the Conventional Model and VSC Model, were assigned to the braces according to the procedure outlined in Section 6.4.3. In addition, plastic hinge locations were assigned at the mid-span of all beams in braced bays.

![Bilinear Spring Properties in VSC Chevron Braced Frame Model](image)

Figure 7.3.1 Bilinear Spring Properties in VSC Chevron Braced Frame Model

The lateral loading pattern applied to the structure for the pushover analysis followed a triangular distribution over the height of the building, using the same proportions as calculated in the seismic design example [Becker et al., 1988] (Table 7.3.1).
Table 7.3.1 Lateral Loading Pattern for Pushover Analysis

<table>
<thead>
<tr>
<th>Floor Level</th>
<th>Elevation (ft)</th>
<th>Elevation (m)</th>
<th>Portion of Total Lateral Force</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>83.0</td>
<td>25.3</td>
<td>0.203</td>
</tr>
<tr>
<td>7</td>
<td>71.5</td>
<td>21.8</td>
<td>0.222</td>
</tr>
<tr>
<td>6</td>
<td>60.0</td>
<td>18.3</td>
<td>0.187</td>
</tr>
<tr>
<td>5</td>
<td>48.5</td>
<td>14.8</td>
<td>0.151</td>
</tr>
<tr>
<td>4</td>
<td>37.0</td>
<td>11.3</td>
<td>0.115</td>
</tr>
<tr>
<td>3</td>
<td>25.5</td>
<td>7.8</td>
<td>0.079</td>
</tr>
<tr>
<td>2</td>
<td>14.0</td>
<td>4.3</td>
<td>0.043</td>
</tr>
</tbody>
</table>

7.4 Analysis Results

The results of the pushover analysis for the two chevron braced frame models are presented in the sections that follow. The response parameters of interest include the frame load-displacement relationship, with particular emphasis on the lateral load capacity of the frame, the maximum roof displacement and the condition of the floor beams in braced bays at the end of the analysis. It should be noted that the results of these analyses are intended for direct comparison purposes only, and should not be used as an evaluation of the design of the structure to current code provisions.

7.4.1 Conventional Chevron Braced Frame Model

The deformed shape of the conventional frame is shown in Figure 7.4.1. It can be seen that the braces at a number of storeys experienced compression buckling during the analysis, some to a greater degree of yielding than others, and that a number of floor beams also suffered large flexural deformations or yielding at mid-span. The load-displacement pushover curve shown in Figure 7.4.2 indicates that linear-elastic behaviour continued until the load reached 5470 kN at a lateral deformation of 0.146 m at the top of the frame. The frame displacement at the roof level reached a maximum value of 0.240 m at a load level of 8525 kN. The maximum inter-storey drift occurred between floors three and four, with a differential displacement of 0.094 m, corresponding to approximately 2.7% of the storey height.
Figure 7.4.1 Conventional Chevron Braced Frame Final Deformed Shape

Figure 7.4.2 Conventional Chevron Braced Frame Pushover Curve
The analysis terminated when the fourth floor braced beam failed in bending. There appears to be an abrupt end to the effectiveness of the bracing system, as Figure 7.4.2 indicates high stiffness just prior to the final displacement step. This frame response could be attributed to the sudden reduction in stiffness at plastic hinge locations, stemming from the representation of the hinge mechanism in the program. Hinge behaviour is nearly rigid-plastic, which means that the hinges have infinite stiffness until the plastic moment is reached, then almost zero stiffness immediately after.

It is important to look more closely at the overall behaviour of the beams in braced bays, as some have undergone flexural yielding, one to the point of failure (as defined by the plastic hinge properties in Section 6.4.3). The beam located in the braced bay at the fourth floor level has experienced a net downward force at mid-span due to the vertical load imbalance between the tension and compression braces below, subsequent to compression brace buckling. As a result, while the pushover analysis continued, the force in the tension brace increased and the moment in the beam grew until it reached the ultimate capacity of the member. Therefore, the integrity of the gravity load system has become questionable, as the load carrying capacity of a beam has been compromised. The tension brace in this bay has become ineffective and the fourth floor will most likely collapse. However, this does not mean that the lateral load resisting system has completely failed, as the columns in the third storey still provide some sway stiffness.

### 7.4.2 VSC Chevron Braced Frame Model

The deformed shape of the VSC frame is shown in Figure 7.4.3. The only non-linear behaviour in the building occurred in the form of brace yielding and compression buckling at a number of storeys during the analysis. The load-displacement pushover curve shown in Figure 7.4.4 indicates that linear-elastic behaviour continued until the load reached 6582 kN at a roof lateral deformation of 0.242 m. The displacement at the top of the frame reached a maximum value of 0.283 m at a load level of 7734 kN. The maximum inter-storey drift occurred between floors three and four, with a differential displacement of 0.056 m, corresponding to approximately 1.6% of the storey height.
Chapter 7

Illustrative Example

Figure 7.4.3 VSC Chevron Braced Frame Final Deformed Shape

Figure 7.4.4 VSC Chevron Braced Frame Pushover Curve
Chapter 7

Illustrative Example

The analysis terminated when a third-storey brace failed. The sudden end of the effectiveness of the bracing system could be due to a significant reduction in stiffness at the plastic hinge locations, described in detail above (Section 7.4.1).

It is important to note that the vertical slotted connection performed effectively during the analysis, as all the floor beams in the braced bays of the VSC Model were protected. While the beams were not damaged in the VSC Model, the vertical displacement at the top of the braces is a critical parameter that needs to be monitored to ensure that the motion does not exceed the maximum permitted bolt travel in the slotted connections. As designed, the slots were sufficient to provide adequate vertical translation in the brace-to-beam connection without imposing unwanted downward forces on the beam. There was slightly more lateral displacement overall in the VSC Model than in the Conventional Model. This is due to the presence of the bilinear link element included in the VSC frames, introduced for calibration purposes in the simplified model (Section 6.4.2). The softening effect on the total stiffness of the frame from the link elements can be seen clearly in the initial portion of the load-displacement curve in Figure 7.4.4.

This illustrative example has demonstrated how a designer can use a commercially available software program to model the proposed VSC chevron braced frame within a real building model, and in its simplicity, has also clearly highlighted the main differences and benefits of the VSC system. The performance of the conventional chevron braced frame reaffirmed the vulnerability of the gravity load resisting system when the lateral load resisting frame is subjected to severe lateral deformations. The floor beams in the Conventional Model showed signs of undesirable deformations at mid-span, yielding and complete failure in the pushover analysis. On the other hand, the behaviour of the VSC chevron braced frame solidified the concept that brace yielding in the lateral load resisting system was achievable without compromising the supporting beams in the gravity framing system. Similar load capacities were achieved with the VSC Model when compared to the Conventional Model. Increased lateral displacements were seen with the VSC frame, due to the presence of the slotted brace-to-beam connection and its softening effect on the overall stiffness of the system.
8.1 Summary

It has been observed in past earthquakes and demonstrated in laboratory testing that chevron braced frames often exhibit poor performance under severe lateral load demands. The problem occurs when the compression brace in the system buckles, causing a vertical force imbalance in the system. As the lateral load on the frame is increased, the force in the tension brace continues to build and a net downward force is transferred to the floor beam above. This concentrated loading at mid-span of the beam can have strong detrimental effects, ranging from large vertical deformations of the beam to yielding and complete failure of the member. Such a response could compromise the gravity load resisting system of a structure and provide a mechanism for its progressive collapse.

There is currently a need in the engineering community in Canada to develop cost-effective design solutions for nominally ductile concentrically braced frames (NDBFs) for low- and medium-rise buildings. Current code provisions require a nominal brace connection force equal to the full tensile yield load of the member \( A_g F_y \) and these forces propagate throughout the design of the structure down to the foundations. This is a costly connection detail that has lead to research efforts to demonstrate that some NDBF systems will not develop the full tensile yield load in the braces, particularly compression buckling systems or braces with reduced net sections.

As a result of these design needs, the vertical slotted connection (VSC) chevron bracing system was developed as a possible viable alternative. The intent of the VSC detail was to keep the force in the braces limited to the compressive resistance of the members by providing no vertical load transfer from the braces to the beam. This would limit the connection forces while also protecting the floor beam from damage. The research on the VSC system described here included both experimental and analytical studies, which were aimed to examine the behaviour of the system under quasi-static cyclic loading.
Chapter 8  Conclusions and Recommendations

conditions, and to determine the suitability of the system for use in high-risk seismic zones.

The experimental program consisted of two quasi-static cyclic tests on VSC chevron bracing specimens. The first test was conducted with hollow tube braces and the second test was conducted with concrete-filled tube braces. The input excitation for both tests was a cyclic displacement time-history applied at a slow loading rate. The results of the testing offered insight into the bracing behaviour and overall system non-linear response.

The analytical study was aimed at establishing a model of the VSC system, calibrated to the observed experimental results, which would serve as a tool for predicting the behaviour and response of the system under various loading conditions. A model replicating the experimental testing was created with two different computer programs using time-history and pushover analyses to compare the results with the experimental data. The analytical study offered further insight into the complexity of the system behaviour and the most accurate ways to represent details of the VSC design. The study also served as a useful tool to compare the ability of different modelling software to replicate the response of the VSC system. The results of this investigation acted as a basis for a more complicated building system illustrative example, providing an evaluation of the model in a more realistic structure.

8.2 Conclusions

The main conclusions from this study are:

- The chevron braces were the sole energy dissipation elements in the braced frame system during cyclic testing. Brace yielding occurred in the form of out-of-plane buckling and plastic hinge formation at the gusset plates at the ends of the braces and at the mid-span of the braces.
- The VSC detail allowed unrestricted vertical movement of the braces throughout the entire loading history of the frame. The length of travel in the slots was just
sufficient to accommodate the full range of brace vertical deformations, as the VSC bolts did not show signs of bearing on the slots until the final stages of testing. No signs of jamming between the gusset plates in the slotted connection were evident.

• The load-displacement curves for both specimens showed the typical characteristics of tension-compression systems, where the brace elements were subjected to compression buckling. The experimental values for the peak lateral load were within 2% of the analytical estimates. However, the measured yield displacements were approximately 165% (16 mm) greater than the predicted values. The primary reason for this discrepancy is believed to result from deformations in the connections, especially the slotted connection. It can be concluded that reasonable estimates of the yield load can be expected. In the calculation of displacements, caution is advised and allowance must be made to accommodate connection flexibilities.

• The VSC detail created a moderate reduction in the overall lateral stiffness of the braced frame system. At large lateral displacements, the top of the braces experienced an average of 7 to 10 mm less movement than the beam above, which was due to the transfer of forces and displacements through the slotted connection. The added flexibility in the VSC detail can be attributed to factors such as lateral slip in the slotted bolted connection, rotation in the slotted bolted connection and out-of-plane movement of the braces between the slotted beam connection plates.

• The concrete-filled tube (CFT) specimen sustained higher peak loads and exhibited superior residual strength throughout testing when compared to the hollow tube (HSS) specimen. The degree of strength and stiffness degradation was also less severe in the CFT specimen than in the HSS specimen.

• The deformed shape for the buckled HSS braces was distinctly bilinear, while the CFT braces showed a more curvilinear buckling shape. This was due to the degree of concentration of local buckling in the braces, as the CFT specimen distributed the effects of local buckling over a long region of the member at mid-span, while, in contrast, the HSS braces were subjected to severe local buckling in a
concentrated region at the mid-span of the member. Premature fracture of the HSS tube walls at this location lead to cracking and eventual tensile failure of the brace cross-section.

- The CFT specimen was able to resist a much larger number of inelastic cycles before fracture and failure than the HSS specimen. Almost six times more energy was dissipated in the CFT system due to the higher residual strength of braces and a greater number of effective load reversal cycles.

- The steel VSC chevron braced frame has been shown to exhibit stable, predictable behaviour under cyclic loading and is a viable concept for lateral load resisting systems in framed structures. Due to the economy and ease of construction of the system, as well as the favourable performance under cyclic loading, it is recommended that the VSC system would be suitable for use in buildings in high-risk seismic zones.

- The introduction of a bilinear spring element in series between the beam and the top of the braces proved to be an effective way to model the VSC element and its effect on the lateral movement of the braces in relation to the beam above. The spring element allowed free movement between the braces and the beam in the vertical direction, but provided a prescribed amount of force and displacement transfer in the system, modelled after the observed experimental response.

- The RUAUMOKO model (with a displacement time-history input similar to that used in testing) provided time-history analysis results showing a reasonable match with the observed experimental response in terms of strength and stiffness in the linear range, and general post-buckling behaviour in the non-linear range. However, the program was unable to provide an adequate representation of the level of energy dissipation for the VSC system, as the hysteretic curves generated by RUAUMOKO gave a gross underestimation of the energy dissipating capacity observed in testing.

- The SAP2000 model provided pushover analysis results showing a good match with the backbone curve from the experimental results. The use of a simplified model with plastic hinges specified at the mid-span and end locations on the
braces proved to be adequate to capture the key stiffness, strength and ductility characteristics of the VSC system.

• In the illustrative example, the SAP2000 simplified model of the VSC chevron braced frame served to illustrate the improved performance of the building system. The VSC Model demonstrated yielding of brace elements with the protection of the beams in braced bays, while the Conventional Model showed large vertical deformations in beams at mid-span and complete failure of a floor beam in one storey. This example provided further evidence of the vulnerability of conventional chevron bracing systems and the benefits of the VSC system.

8.3 Recommendations

Preliminary experimental tests have shown a great deal of promise for the VSC chevron brace system; however, further experimental studies would provide valuable information to complement these test results. Recommendations for further research include:

• Component testing of the VSC detail under cyclic quasi-static loading conditions, with particular emphasis on identifying the relationship between the slip in the VSC detail, the torque of the bolts in the slotted connection and the yield displacement value. This detailed study could provide an indication of the sensitivity of the yield displacement to the slip between the bolts and slotted holes or the slip between the gusset plates in the VSC detail.

• Additional full-scale quasi-static cyclic testing with modified loading histories and specimen design details. These studies would provide further opportunities to observe the response of other innovative design details under various loading conditions. A benchmark test with a conventional chevron braced frame should be included in the scope of testing of any sort for comparative purposes.

• Dynamic shake table testing under a variety of seismic input excitations, including subduction and crustal earthquake records as well as records suitable for studying the near-field effects of earthquakes.
Chapter 8 Conclusions and Recommendations

- Detailed finite element modelling (FEM) of the VSC chevron braced frame to better understand the slotted bolted connection behaviour and to study the brace buckling mechanism more closely, particularly the local buckling effects at the mid-span of the braces. Additionally, the effects of concrete fill should be included in detailed FEM efforts.

- Three-dimensional modelling of complete building systems incorporating the VSC chevron braced frame. These studies would make it possible to capture the full load redistribution effects in buildings and help to better predict the system response to earthquake excitations.

The VSC chevron braced frame research also has important implications for current engineering practice. Recommendations for practical use of the VSC system include:

- Appropriate design and detailing for chevron braces in NDBFs to permit brace yielding and plastic hinge formation at the desired locations along the brace elements.

- Sizing of braces to meet slenderness requirements and facilitate desirable behaviour in non-linear range following global buckling.

- Use of concrete fill in tubular members to increase the strength and fracture life of braces and promote more favourable ductility and energy dissipation characteristics.

- Detailing of the VSC and chevron braces to permit bolted field connections to the surrounding frame wherever possible. This will encourage the use of the system both in new construction and in retrofit schemes.

- Detailing of the VSC to allow the proper tolerances for fit-up, but to avoid excessive slip and rotation to be permitted in slotted bolted connection.

- Specification for installation of bolts in the slotted connection to be fastened only "finger tight" unless the design concept has accounted for additional frictional resistance from the slotted bolted connection.

- Inclusion of appropriate joint constraints in analytical models to accurately represent the observed behaviour of the VSC chevron braced frame during cyclic
testing. The use of a spring element in series between the beam and the braces was found to be an effective way to account for the flexibility in the restraint of the VSC detail without using a more detailed FEM approach for the system.

- Consideration in code provisions for reducing the nominal force requirement for bracing connections to the governing load in the bracing system. For the VSC chevron braced frame, it has been demonstrated that the compressive resistance, not the tensile resistance of the member, determines the maximum force in the braces. The design force should also include appropriate safety factors, for accepted uncertainties associated with loading and resistance, as well as consideration for material overstrength.
REFERENCES


SAC. 1995. Steel Moment Frame Connection – Advisory No. 3. SAC 95-01, SAC Joint Venture, Sacramento, California.

Steinbrugge, K.V. 1971. Karl V. Steinbrugge Collection, Earthquake Engineering Research Center, University of California, Berkeley, California.


APPENDIX A – INSTRUMENTATION DETAILS
Transducer Type: LVDT
Brand Name: SENSOTEC
Model: 060-3615-02
Serial Number: L0349201
Range: ±1.000 inch
Supply: 6.0 VDC
Location Used: Stub Column Testing

Transducer Type: LVDT
Brand Name: SENSOTEC
Model: 060-3615-02
Serial Number: L0332306
Range: ±1.000 inch
Supply: 6.0 VDC
Location Used: Stub Column Testing

Transducer Type: Strain Gauge
Brand Name: TML
Model: QFLA-3:350-11
Lot Number: A507412
Gauge Length: 3 mm
Gauge Resistance: 350 ± 1.0 Ω
Gauge Factor: 2.12 ± 1%
Location Used: Stub Column Testing

Transducer Type: LVDT
Brand Name: SENSOTEC
Range: ±0.500 inch
Location Used: Material Testing

Transducer Type: Load Cell
Brand Name: MTS
Model: 661.22
Serial Number: 347
Location Used: VSC Chevron Brace Testing, Channel 1

Transducer Type: Displacement String-Potentiometer
Brand Name: Celesco
Model: PT101-0025-111-1110
Serial Number: D1001625
Sensitivity: 36.900 mV/V/inch
Location Used: VSC Chevron Brace Testing, Channel 3

Transducer Type: Displacement String-Potentiometer
Brand Name: Celesco
Model: PT101-0025-111-1110
Serial Number: D1001623
Sensitivity: 36.876 mV/V/inch
Location Used: VSC Chevron Brace Testing, Channel 32

Transducer Type: LVDT
Brand Name: SENSOTEC
Model: 060-3615-02
Serial Number: L0349201
Range: ±1.000 inch
Supply: 6.0 VDC
Location Used: VSC Chevron Brace Testing, Channel 5

Transducer Type: LVDT
Brand Name: SENSOTEC
Model: 060-3615-02
Serial Number: L0332306
Range: ±1.000 inch
Supply: 6.0 VDC
Location Used: VSC Chevron Brace Testing, Channel 4

Transducer Type: Strain Gauge
Brand Name: TML
Model: QFLA-3-350-11
Lot Number: A507412
Gauge Length: 3 mm
Gauge Resistance: 350 ± 1.0 Ω
Gauge Factor: 2.12 ± 1%
Location Used: VSC Chevron Brace Testing, Channel 8-19

Transducer Type: Strain Gauge
Brand Name: TML
Model: FCA-3-350-11
Lot Number: A510421
Gauge Length: 3 mm
Gauge Resistance: 350 ± 1.0 Ω
Gauge Factor: 1=2.13 ± 1%; 2=2.13 ± 1%
Location Used: VSC Chevron Brace Testing, Channel 6-7

Transducer Type: Strain Gauge
Brand Name: TML
Model: FRA-3-350-11
Lot Number: A510431
Gauge Length: 3 mm
Gauge Resistance: 350 ± 1.0 Ω
Gauge Factor: 1=2.12 ± 1%; 2=2.12 ± 1%; 3=2.12 ± 1%
Location Used: VSC Chevron Brace Testing, Channel 20-31
APPENDIX B - FABRICATION DRAWINGS
TEST FRAME DETAILS

ACTUATOR DETAIL

~12"

~8 1/2"

(12)

OCT. 17/00
R. BUBELA

(SKETCHES N.T.B.)

TEST FRAME ELEVATION

SECTION A-A

DETAIL 1

SECTION B-B
### BILL OF MATERIAL

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**SOLID ROCK STEEL FABRICATING CO.**

**MATERIAL:** HSS=89K (BANDSPLICE=40)

**HOLES:** 1/4" UNLESS NOTED

**FINISH:** PLAIN, NO PAINT

**FIELD BOLTS:**

| 24  | 1/2" A325 X 3 1/2 |
| 24  | 1/2" A325 X 3 1/2 |
| 8   | 5/8" A325 X 3 1/2 |

**TITLE:** DETAILS TEST FRAME

**CUSTOMER:** C.I.S.C.

**PROJECT:** TEST FRAME

**LOCATION:** VANCOUVER

**DRAWN BY:** NAV NOV 2000

**CHECKED BY:** SJ DEC 2000

**CONTRACT:** 1353

**REV.:** 01
APPENDIX C – TEST PHOTOGRAPHS
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Table C.3 Test 2 (to Failure) Photo Catalogue

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<td>8</td>
<td>18</td>
<td>+150 mm</td>
<td>South brace local buckling</td>
</tr>
<tr>
<td>Figure C.88</td>
<td>8</td>
<td>18</td>
<td>-150 mm</td>
<td>North brace local buckling</td>
</tr>
<tr>
<td>Figure C.89</td>
<td>8</td>
<td>18-19</td>
<td>0 mm</td>
<td>South brace visible cracking</td>
</tr>
<tr>
<td>Figure C.90</td>
<td>8</td>
<td>19</td>
<td>+150 mm</td>
<td>North brace tension cracking</td>
</tr>
<tr>
<td>Figure C.91</td>
<td>8</td>
<td>19</td>
<td>-150 mm</td>
<td>North brace local buckling</td>
</tr>
<tr>
<td>Figure C.92</td>
<td>8</td>
<td>20</td>
<td>+150 mm</td>
<td>North brace tension cracking</td>
</tr>
<tr>
<td>Figure C.93</td>
<td>8</td>
<td>20</td>
<td>+150 mm</td>
<td>South brace local buckling</td>
</tr>
<tr>
<td>Figure C.94</td>
<td>8</td>
<td>20</td>
<td>-150 mm</td>
<td>South brace tension cracking</td>
</tr>
<tr>
<td>Figure C.95</td>
<td>8</td>
<td>20</td>
<td>-150 mm</td>
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</tr>
<tr>
<td>Figure C.96</td>
<td>8</td>
<td>21</td>
<td>+150 mm</td>
<td>North brace tension cracking</td>
</tr>
<tr>
<td>Figure C.97</td>
<td>8</td>
<td>23</td>
<td>0 mm</td>
<td>Specimen failure</td>
</tr>
<tr>
<td>Figure C.98</td>
<td>8</td>
<td>23</td>
<td>0 mm</td>
<td>Specimen failure</td>
</tr>
</tbody>
</table>
APPENDIX D – NUMERICAL PREDICTION OF TEST SPECIMEN BEHAVIOUR
Chevron Brace Project - Numerical Prediction

Brace Geometry:

\[
L_{\text{brace}} = 3147 \text{ mm} \quad \text{(brace length)}
\]

\[
L_{\text{strain}} = 2807 \text{ mm} \quad \text{(brace between gussets)}
\]

Width := 2921 mm

Height := 3355 mm

\[
L_{\text{wp}} = \sqrt{\left(\frac{\text{Width}}{2}\right)^2 + \text{Height}^2} \quad L_{\text{wp}} = 3659 \text{ mm}
\]

\[
\theta = \tan^{-1}\left(\frac{\text{Height}}{0.5 \times \text{Width}}\right) \quad \theta = 1
\]

\[
\theta_{\text{deg}} = \frac{180}{\pi} \theta_{\text{deg}} = 66
\]

Load and Displacement Control:

Controlled Lateral Displacement:

\[
u = 9.6 \text{ mm}
\]

Brace Deformation:

\[
\delta := u \cdot \cos(\theta) \quad \delta = 3.83 \text{ mm} \quad \text{(brace shortening)}
\]

Forces:

\[
P := \frac{E \cdot A}{L_{\text{wp}}} \delta \quad P = 325 \text{ kN} \quad \text{(brace buckling load)}
\]

\[
V := 2 \cdot P \cdot \cos(\theta) \quad V = 259 \text{ kN} \quad \text{(frame lateral load at brace buckling)}
\]

Stresses and Strains:

\[
\sigma := \frac{P}{A} \quad \sigma = 209 \text{ MPa}
\]

\[
\varepsilon := \frac{\sigma}{E} \quad \varepsilon = 1.05 \times 10^{-3}
\]
Brace Member Capacity:

1) Compression - Euler Buckling

\[ K := 0.90 \]
\[ I = 1.79 \times 10^{-6} \text{ m}^4 \]
\[ L_e := K \cdot L_w \]
\[ L_e = 3.293 \text{ m} \]
\[ P_e := \frac{\pi^2 \cdot E \cdot I}{L_e^2} \]
\[ P_e = 326 \text{ kN} \]

2) Tension - Brace Yielding

\[ \phi := 0.9 \]
\[ T_r := \phi \cdot A \cdot F_y \]
\[ T_r = 635 \text{ kN} \]
\[ T := \frac{T_r}{\phi} \]
\[ T = 705 \text{ kN} \]

Member "Strength" Limits: (material strength or stability)

\[ P_{cr(sl)} := \frac{\pi^2 \cdot E \cdot A}{sl^2} \]
\[ T_{cr(sl)} := A \cdot F_y \]
\[ SL := \frac{K \cdot L_w}{r} \]
\[ SL = 97 \]

Critical Force vs. Slenderness