FORK CONFIGURATION DAMPERS (FCDs)
FOR ENHANCED DYNAMIC PERFORMANCE
OF HIGH-RISE BUILDINGS

by

Michael Montgomery

A thesis submitted in conformity with the requirements
for the degree of Doctor of Philosophy
Department of Civil Engineering
University of Toronto

© Copyright by Michael Montgomery (2011)
ABSTRACT

Fork Configuration Dampers (FCDs) for Enhanced Dynamic Performance of High-Rise Buildings

Doctor of Philosophy (2011)

Michael Montgomery

Department of Civil Engineering

University of Toronto

The dynamic behaviour of high-rise buildings has become a critical design consideration as buildings are built taller and more slender. Large wind vibrations cause an increase in the lateral wind loads, but more importantly, they can be perceived by building occupants creating levels of discomfort ranging from minor annoyance to severe motion sickness. The current techniques to address these issues include stiffening the lateral load resisting system, reducing the number of stories, or incorporating a vibration absorber at the top of the building. All of which have consequences on the overall project cost. The dynamic response of high-rise buildings is highly dependent on damping. Full-scale measurements of high-rise buildings have shown that the inherent damping decreases with height and recent in-situ measurements have shown that the majority of buildings over 250 meters have levels of damping less than 1% of critical. Studies have shown that small increases in the inherent damping can lead to vast improvement in dynamic response. A new damping system, the viscoelastic (VE) Fork Configuration Damper (FCD), has been developed at the University of Toronto to address these design challenges. The proposed FCDs are introduced in lieu of coupling beams in reinforced concrete (RC) coupled wall buildings and take advantage of the large shear deformations at these locations when the building is subjected to lateral loads. An experimental study was conducted on 5 small-scale VE dampers to characterize the VE material behaviour and 6 full-scale FCD samples in an RC coupled wall configuration (one designed for areas where low to moderate ductility is required and one with a built-in ductile structural “fuse” for areas where high ductility is required). The VE material tests exhibited stable hysteretic behaviour under expected high-rise loading conditions and the full-scale tests validated the overall system performance based on the kinematic behaviour of coupled walls, wall anchorage and VE material behaviour. Analytical models were developed that capture the VE material behaviour and the FCD system performance well. An 85-storey high-rise building was studied analytically to validate the design approach and to highlight the improvements in building response resulting from the addition of FCDs.
ACKNOWLEDGEMENTS

I am indebted to my supervisor Professor Constantin Christopoulos for giving me the confidence, direction and encouragement to conduct this study. I believe his excellent mentorship directly relate to the quality of the research. His enthusiasm to explore new areas in engineering research in a logical and practical manner have inspired me. I look up to him both personally and professionally and hope I may be able to follow his example. Working with him has been exciting, challenging and rewarding and I believe it has resulted in significant growth of me personally. Thank you.

I would like to extend a sincere thank you to Professor Michael Collins. It was truly an honour to have him on my thesis committee and I appreciate all of the work and time he put into reviewing the thesis. I have benefited greatly from listening to his elegant explanations of engineering concepts over the years.

A special thanks to Professor Evan Bentz for all of the interesting discussions and also his willingness to provide advice and suggestions in a very humble manner.

I would also like to thank Dr. Ian Aiken for his thorough and thoughtful review of the thesis.

Thank you to Professor Robert Tremblay for facilitating access to the Laboratoire de Structures Hydro-Quebec at École Polytechnique, for such a large testing program. Also, thanks to Professor Pierre Leger for welcoming the “grandsons” to Montreal and making us feel like part of a big extended family.

Thank you to Halcrow Yolles for their support of this project and for a door into the world of high-rise building design. A special thank you to the excellent structural engineers at Halcrow Yolles that were directly involved during the early stages of the project: Tibor Kokai, Agha Hasan and Sean Smith. Working on the challenges related to the design of real high-rise buildings with them was extremely instructive, rewarding and satisfying.

The test specimens were fabricated and delivered in-kind by Nippon Steel Engineering Corporation (NSEC). The many iterations with NSEC engineers were extremely important in order to develop a constructible and economic FCD. Their contributions to design and quality control were invaluable to the project.

The funding and financial support provided throughout the thesis were provided by the Natural Sciences and Engineering Research Council of Canada (NSERC) under the Idea to Innovation (I2I) Program and the Industrial Postgraduate Scholarship Program (IPS). This financial commitment was crucial for the completion of my project.
Reinforced concrete test specimens were fabricated by Béton Brunet in Valleyfield, Québec. Special thanks to Martin Montpetit for his willingness to work on research projects in areas not directly associated with precast concrete. Structural steel was supplied by CanAM in Boucherville, Québec. I would especially like to thank Pierre Gignac for his work in resolving some details with the pins. Additional thanks to Tom Cerreto from ERICO for contributing the LENTON weldable couplers to the project.

Thank-you to all the laboratory staff at University of Toronto and Ecole Polytechnique in Montreal. In Toronto I would like to especially thank Renzo Basset and John MacDonald for patience and work on the sets of tests, as well as Giovanni Buzzeo and Xiaoming Sun, for lending a helping hand. In Montreal I would like to thank Guillaume Cossette, Viacheslav Koval, Marc Charbonneau for their dedication to my large test program and also Denis Fortier, Patrice Bélanger, David Ek and Martin Leclerc for valuable suggestions in the lab. Also, I would like to thank Anne-Marie Goulet for easing the transition to Montreal for me.

One of the many things I will miss about grad school is the interaction, technical and otherwise, shared with my close colleagues and friends. In particular I would like to thank Jeff Erochko, Lydell Wiebe, Michael Gray, Carlos de Oliveira, Nabil Mansour, Jack Guo, Boyan Mihailov, Kien Vinh Duong, amongst many others. Over the years they helped me understand many engineering concepts and we have become great friends.

I would like to thank my close family and friends as well as my extended family. Especially, my mother for her unwavering love; my father for his passion for structural engineering and for being an example of a man I wanted to become since childhood; my brother for his companionship through the years and my grandparents for their support for wherever life takes me.

Finally I would like to thank Susi who has been understanding and loving through the most hectic portions of my Ph.D. She has been a guiding light and a pillar for me.
# TABLE OF CONTENTS

**ABSTRACT**........................................................................................................................................... ii

**ACKNOWLEDGEMENTS**.................................................................................................................. iii

**TABLE OF CONTENTS** ...................................................................................................................... v

**LIST OF FIGURES** .......................................................................................................................... viii

**LIST OF TABLES** ............................................................................................................................ xiii

**LIST OF SYMBOLS** ........................................................................................................................... xv

**CHAPTER 1: INTRODUCTION** .............................................................................................................1

1.1 Background and Motivation .............................................................................................................1

1.2 Objective of Work ........................................................................................................................... 4

1.3 Scope of Work .................................................................................................................................. 7

**CHAPTER 2: BEHAVIOUR AND DESIGN OF RC COUPLED WALL HIGH-RISE BUILDINGS** ................. 8

2.1 Introduction ........................................................................................................................................ 8

2.2 Introduction to RC Coupled Wall Buildings ................................................................................... 9

2.2.1 Modelling of RC Coupled Wall Buildings .................................................................................. 9

2.2.2 Inherent Damping Properties of RC Buildings .......................................................................... 21

2.2.3 Measured Damping and Frequency Values of High-Rise Buildings ........................................... 24

2.3 High-rise Buildings Subject to wind ................................................................................................. 30

2.3.1 Introduction .................................................................................................................................. 30

2.3.2 Random Vibration Approach ..................................................................................................... 30

2.3.3 Wind Characteristics .................................................................................................................... 31

2.3.4 Wind Tunnel Model Testing ....................................................................................................... 38

2.3.5 High-frequency Load Balance Test ............................................................................................. 39

2.3.6 High-Frequency Force Balance (HFFB) Procedure ...................................................................... 40

2.3.7 Dependency of High-Rise Building Response on Dynamic Properties ........................................ 45

2.4 Wind Design Philosophy .................................................................................................................. 48

2.4.1 Service Limit State Design Strategy ........................................................................................... 49

2.4.2 Ultimate Limit State Design Strategy .......................................................................................... 53

2.4.3 Design of Walls ............................................................................................................................ 54

2.4.4 Design Procedure ........................................................................................................................ 54

2.5 Seismic Design Philosophy ............................................................................................................. 55

2.5.1 Seismic Behaviour of Coupled Wall Buildings .......................................................................... 55

2.5.2 Review of Research on Cyclic Behaviour of Coupling Beams ..................................................... 57

2.5.3 Design Guidelines ......................................................................................................................... 59

2.5.4 Design Procedure ........................................................................................................................ 61

2.6 Current Methods to Mitigate Lateral Vibrations .............................................................................. 63

2.6.1 Passive Dynamic Vibration Absorbers ......................................................................................... 63

2.6.2 Tuned Liquid Dampers: Tuned Sloshing Dampers (TSDs) and Tuned Liquid Column Dampers (TLCDs) ......................................................................................................................................................... 68

2.7 Comparison of Wind Design and Seismic Design ........................................................................... 71

2.8 Conclusions ....................................................................................................................................... 72
**CHAPTER 3: FORK CONFIGURATION DAMPERS (FCDs) FOR USE IN RC COUPLED WALL HIGH-RISE BUILDINGS** ...............................................73

3.1 Current Challenges in High-Rise Design and Need For Added Damping .......... 73
3.2 VE Fork Configuration Dampers (FCDs) .............................................................. 75
   3.2.1 Evolution of FCD ............................................................................................. 76
3.3 Additional FCD Details and Designs ................................................................. 81
   3.3.1 Slab Connection Details .............................................................................. 81
   3.3.2 Wall Connection Details ............................................................................. 82
   3.3.3 Structural “Fuse” Details ............................................................................. 84
   3.3.4 Replaceable FCD Details ............................................................................. 84
   3.3.5 Outrigger FCD .............................................................................................. 85
   3.3.6 Additional FCD Structural Configurations .............................................. 87
3.4 Viscoelastic Material ............................................................................................. 87
   3.4.1 Hysteretic Behaviour of VE Material ....................................................... 87
   3.4.2 Past Applications ....................................................................................... 90
   3.4.3 Equivalent VE Modelling Technique for FCD .......................................... 92
   3.4.4 Viscoelastic-Plastic Modelling Techniques for FCD ............................... 95
   3.4.5 Optimization of FCD Locations ............................................................... 96
3.5 Conclusion ............................................................................................................. 97

**CHAPTER 4: EXPERIMENTAL VALIDATION OF FORK CONFIGURATION DAMPERS (FCDs) ......................................................98

4.1 Introduction ........................................................................................................... 98
4.2 Small-scale Damper and Viscoelastic Material Characterization at University of Toronto (UofT) ............................................................... 98
   4.2.1 Test Configuration and Damper Design .................................................. 98
   4.2.2 Loading Protocol ....................................................................................... 100
   4.2.3 Small-Scale Test Results .......................................................................... 105
   4.2.4 Small-Scale Test Conclusions ................................................................. 117
4.3 Full-Scale Fork Configuration Damper Tests at Ecole Polytechnique Montreal (EPM) .............................................................. 118
   4.3.1 Design of Test Specimen FCD-A ............................................................ 118
   4.3.2 Design of Test Specimen FCD-B ............................................................ 125
   4.3.3 Full-Scale Test Design ............................................................................ 130
   4.3.4 Test Instrumentation ................................................................................ 137
   4.3.5 Material Tests .......................................................................................... 142
   4.3.6 Test Summary and Loading Protocol ...................................................... 143
   4.3.7 Full-Scale FCD Test Results ................................................................. 147
   4.3.8 FCD Design and Modelling Properties ............................................... 190
   4.3.9 Full-Scale Test Conclusions ................................................................. 194

**CHAPTER 5: NUMERICAL MODELLING OF FORK CONFIGURATION DAMPERS (FCDs) .................................................................195

5.1 Overview of Development of FCD Models ...................................................... 195
5.2 VE Material Modelling ...................................................................................... 195
   5.2.1 Kelvin-Voigt Model (KVM) .................................................................... 196
   5.2.2 Generalized Maxwell Model (GMM4) ..................................................... 197
   5.2.3 VE Parameter Estimation ........................................................................ 204
   5.2.4 Comparison of the KVM and GMM4 with Small-Scale Test Results ...... 205
5.3 Modelling Techniques for the FCD Response ................................................. 212
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.3.1</td>
<td>Equivalent VE Model (Model 1)</td>
<td>213</td>
</tr>
<tr>
<td>5.3.2</td>
<td>Spring Kelvin-Voigt Model (Model 2)</td>
<td>214</td>
</tr>
<tr>
<td>5.3.3</td>
<td>Generalized Maxwell Model (Model 3)</td>
<td>216</td>
</tr>
<tr>
<td>5.3.4</td>
<td>FCD Modelling Results</td>
<td>218</td>
</tr>
<tr>
<td>5.4</td>
<td>Further Refinement of Model 3 through Additional Calibration</td>
<td>234</td>
</tr>
<tr>
<td>5.5</td>
<td>Modelling FCDs in Commercial Software</td>
<td>236</td>
</tr>
<tr>
<td>5.6</td>
<td>Modelling Conclusion</td>
<td>236</td>
</tr>
<tr>
<td>5.6.1</td>
<td>Model Implications for Practitioners</td>
<td>237</td>
</tr>
<tr>
<td>5.4</td>
<td>Further Refinement of Model 3 through Additional Calibration</td>
<td>234</td>
</tr>
<tr>
<td>5.5</td>
<td>Modelling FCDs in Commercial Software</td>
<td>236</td>
</tr>
<tr>
<td>5.6</td>
<td>Modelling Conclusion</td>
<td>236</td>
</tr>
<tr>
<td>5.6.1</td>
<td>Model Implications for Practitioners</td>
<td>237</td>
</tr>
<tr>
<td>6.1</td>
<td>Designing High-Rise Buildings Incorporating FCDs</td>
<td>240</td>
</tr>
<tr>
<td>6.2</td>
<td>Wind and Seismic FCD Design Philosophy</td>
<td>240</td>
</tr>
<tr>
<td>6.2.1</td>
<td>Proposed Wind Design Strategy with FCDs</td>
<td>242</td>
</tr>
<tr>
<td>6.2.2</td>
<td>Considered Load Cases</td>
<td>244</td>
</tr>
<tr>
<td>6.2.3</td>
<td>Suggested FCD Design Properties</td>
<td>246</td>
</tr>
<tr>
<td>6.2.4</td>
<td>Wind Design Procedure with FCDs</td>
<td>249</td>
</tr>
<tr>
<td>6.3</td>
<td>Proposed Seismic Design Strategy with FCDs</td>
<td>251</td>
</tr>
<tr>
<td>6.3.1</td>
<td>Proposed Seismic Design Strategy with FCDs</td>
<td>251</td>
</tr>
<tr>
<td>6.3.2</td>
<td>Modelling FCDs for Earthquake Analyses</td>
<td>254</td>
</tr>
<tr>
<td>6.3.3</td>
<td>Seismic Design Procedure with FCDs</td>
<td>258</td>
</tr>
<tr>
<td>6.4</td>
<td>85-Storey Case Study Structure</td>
<td>260</td>
</tr>
<tr>
<td>6.4.1</td>
<td>FCD Solution</td>
<td>264</td>
</tr>
<tr>
<td>6.4.2</td>
<td>Evaluation of Added Damping</td>
<td>266</td>
</tr>
<tr>
<td>6.4.3</td>
<td>ULS Design</td>
<td>274</td>
</tr>
<tr>
<td>6.4.4</td>
<td>Earthquake Time-History</td>
<td>278</td>
</tr>
<tr>
<td>6.4.5</td>
<td>Conclusions from Case Study</td>
<td>278</td>
</tr>
<tr>
<td>7.1</td>
<td>Overview of Thesis</td>
<td>281</td>
</tr>
<tr>
<td>7.2</td>
<td>Comparison of FCD system to Current Vibration Absorbers</td>
<td>283</td>
</tr>
<tr>
<td>7.3</td>
<td>Potential New Design Philosophies and Potential Additional Benefits</td>
<td>285</td>
</tr>
<tr>
<td>7.4</td>
<td>Recommendations for Further Research</td>
<td>286</td>
</tr>
<tr>
<td>289</td>
<td>REFERENCES</td>
<td></td>
</tr>
<tr>
<td>A-1</td>
<td>APPENDIX A: FCD CAPACITY DESIGN CALCULATIONS</td>
<td></td>
</tr>
<tr>
<td>B-1</td>
<td>APPENDIX B: FULL-SCALE SPECIMEN DRAWINGS</td>
<td></td>
</tr>
<tr>
<td>C-1</td>
<td>APPENDIX C: SMALL-SCALE TEST SUMMARY</td>
<td></td>
</tr>
<tr>
<td>D-1</td>
<td>APPENDIX D: FULL-SCALE FCD TESTS AT ECOLE POLYTECHNIQUE IN MONTREAL</td>
<td></td>
</tr>
</tbody>
</table>
**LIST OF FIGURES**

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>RC Coupled Wall Structures</td>
</tr>
<tr>
<td>1.2</td>
<td>Fork Configuration Dampers (FCDs)</td>
</tr>
<tr>
<td>1.3</td>
<td>Kinematics of Buildings Equipped with FCDs</td>
</tr>
<tr>
<td>1.4</td>
<td>Superposition of Bending Stresses of Coupled Walls Subject to Lateral Loads</td>
</tr>
<tr>
<td>1.5</td>
<td>Coupled Wall Subject to Lateral Wind Loads</td>
</tr>
<tr>
<td>1.6</td>
<td>Representation of a Coupled Wall by the Continuum Medium Method</td>
</tr>
<tr>
<td>1.7</td>
<td>Relative Displacement Contributions at Point of Contraflexure</td>
</tr>
<tr>
<td>1.8</td>
<td>RC 40-storey Coupled Wall Structure Subject to Uniform Load</td>
</tr>
<tr>
<td>1.9</td>
<td>Structural Analysis Modelling Techniques</td>
</tr>
<tr>
<td>1.10</td>
<td>Deformed 3-D Coupled Wall Building</td>
</tr>
<tr>
<td>1.11</td>
<td>Jeary's Amplitude Dependent Model for Buildings (after Jeary 1986 and Jeary 1996)</td>
</tr>
<tr>
<td>1.12</td>
<td>Inherent Dynamic Properties of High-Rise Buildings: a, b and c adapted from Satake (2003), d adapted from Li et al. (2003)</td>
</tr>
<tr>
<td>1.13</td>
<td>Random Vibration Approach to Resonant Dynamic Wind Response (adapted from Davenport, 1963)</td>
</tr>
<tr>
<td>1.14</td>
<td>Wind Velocity Profile with Height</td>
</tr>
<tr>
<td>1.15</td>
<td>Wind Power Spectrums</td>
</tr>
<tr>
<td>1.16</td>
<td>Normalized Power Spectral Density and Arbitrary Bluff Body Forces</td>
</tr>
<tr>
<td>1.17</td>
<td>Examples of Irregular Flow and Turbulent Behaviour of Tall Buildings in Wind Tunnel</td>
</tr>
<tr>
<td>1.18</td>
<td>Wind Tunnel Studies</td>
</tr>
<tr>
<td>1.19</td>
<td>Background and Resonant Components of Response in the Along-Wind Direction (adapted from Holmes, 2007)</td>
</tr>
<tr>
<td>1.20</td>
<td>40-Storey Building</td>
</tr>
<tr>
<td>1.21</td>
<td>85-Storey Building</td>
</tr>
<tr>
<td>1.22</td>
<td>Perception Thresholds for Horizontal Vibrations (adapted from Tamura 2006)</td>
</tr>
<tr>
<td>1.23</td>
<td>Design Approach for RC High-Rise Coupled Wall Buildings under Wind Loading</td>
</tr>
<tr>
<td>1.24</td>
<td>Coupling Wall Buildings Subjected to Earthquakes</td>
</tr>
<tr>
<td>1.25</td>
<td>Effect of Load Reversals on Coupling Beam Shear Capacity (after Morawski 1980)</td>
</tr>
<tr>
<td>1.26</td>
<td>Tests of New Diagonal Reinforcing Patterns for Coupling Beams</td>
</tr>
<tr>
<td>1.27</td>
<td>Design Approach for RC High-Rise Coupled Wall Buildings under Earthquake Loading</td>
</tr>
<tr>
<td>1.28</td>
<td>Undamped Main Structure and TMD (Frahm's Absorber)</td>
</tr>
<tr>
<td>1.29</td>
<td>Damped Main Structure and TMD System</td>
</tr>
<tr>
<td>1.30</td>
<td>TMD Applications</td>
</tr>
<tr>
<td>1.31</td>
<td>Tuned Liquid Damper Systems</td>
</tr>
<tr>
<td>1.32</td>
<td>Design Philosophy and Expected Behaviour</td>
</tr>
<tr>
<td>1.33</td>
<td>Coupled Wall Buildings</td>
</tr>
<tr>
<td>1.34</td>
<td>Configuration of FCDs in Coupled Shear Wall Building Application</td>
</tr>
<tr>
<td>1.35</td>
<td>Exaggerated Deformed Shape of RC Coupled Wall Building with FCDs</td>
</tr>
<tr>
<td>1.36</td>
<td>Evolution of FCD Design</td>
</tr>
<tr>
<td>1.37</td>
<td>Proposed FCD Solution for 85-Storey High-Rise in Toronto</td>
</tr>
<tr>
<td>1.38</td>
<td>Proposed FCD Solution for 75-Storey High-Rise in Korea</td>
</tr>
<tr>
<td>1.39</td>
<td>Potential Slab Connection Details</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>3.8</td>
<td>Proposed Connection Details</td>
</tr>
<tr>
<td>3.9</td>
<td>Proposed Construction Sequence using Weldable Couplers</td>
</tr>
<tr>
<td>3.10</td>
<td>Potential “Fuse” Mechanisms</td>
</tr>
<tr>
<td>3.11</td>
<td>Potential Replaceable Link Connections</td>
</tr>
<tr>
<td>3.12</td>
<td>Potential FCD Outrigger Configuration</td>
</tr>
<tr>
<td>3.13</td>
<td>VE Material Characteristics under Harmonic Loading</td>
</tr>
<tr>
<td>3.14</td>
<td>Infinitesimal Element Subject to Pure Shear</td>
</tr>
<tr>
<td>3.15</td>
<td>World Trade Center VE Application</td>
</tr>
<tr>
<td>3.16</td>
<td>Equivalent Viscoelastic Model, adapted from Kasai (2006)</td>
</tr>
<tr>
<td>3.17</td>
<td>FCD Equivalent Viscoelastic Model in Shear</td>
</tr>
<tr>
<td>3.18</td>
<td>FCD Viscoelastic-Plastic Model in Shear</td>
</tr>
<tr>
<td>4.1</td>
<td>5mm Thick SSVD Details</td>
</tr>
<tr>
<td>4.2</td>
<td>10mm Thick SSVD Details</td>
</tr>
<tr>
<td>4.3</td>
<td>SSVD tests</td>
</tr>
<tr>
<td>4.4</td>
<td>FCD SLS Wind Storm Response</td>
</tr>
<tr>
<td>4.5</td>
<td>Wind Loading Protocol</td>
</tr>
<tr>
<td>4.6</td>
<td>Frequency Dependency of VE Material Response at a Strain of 50% for the First Cycle of Loading</td>
</tr>
<tr>
<td>4.7</td>
<td>Frequency Dependence of the Average VE Material Properties for the First Cycle of the Material Characterization Tests</td>
</tr>
<tr>
<td>4.8</td>
<td>Strain Dependency of VE Material Response at a Frequency of 0.2 Hz for the First Cycle of Loading</td>
</tr>
<tr>
<td>4.9</td>
<td>Strain Dependence of the Average Properties from the First Cycle of the Material Characterization Tests</td>
</tr>
<tr>
<td>4.10</td>
<td>Ambient Temperature Dependency of VE Material Response at a Strain of 50% for the First Cycle of Loading</td>
</tr>
<tr>
<td>4.11</td>
<td>Material Characterization Hysteresis: UT-VEM10-1</td>
</tr>
<tr>
<td>4.12</td>
<td>Material Characterization VE Material Properties: UT-VEM10-1</td>
</tr>
<tr>
<td>4.13</td>
<td>Across-wind Hysteresis: UT-VEM5-2</td>
</tr>
<tr>
<td>4.14</td>
<td>Across-wind VE Material Properties: UT-VEM5-2</td>
</tr>
<tr>
<td>4.15</td>
<td>Along-wind Hysteresis: UT-VEM5-2</td>
</tr>
<tr>
<td>4.16</td>
<td>Along-wind VE Material Properties: UT-VEM5-2</td>
</tr>
<tr>
<td>4.17</td>
<td>MCE VE Material Properties: UT-VEM5-2</td>
</tr>
<tr>
<td>4.18</td>
<td>Control Tests: UT-VEM10-1</td>
</tr>
<tr>
<td>4.19</td>
<td>Full-Scale FCD Specimens</td>
</tr>
<tr>
<td>4.20</td>
<td>FCD-A Details</td>
</tr>
<tr>
<td>4.21</td>
<td>FCD Manufacturing Photos</td>
</tr>
<tr>
<td>4.22</td>
<td>FCD-A</td>
</tr>
<tr>
<td>4.23</td>
<td>Weldable Lenton Half Couplers End-Plate Connection</td>
</tr>
<tr>
<td>4.24</td>
<td>FCD-A: Capacity Design Force Definition</td>
</tr>
<tr>
<td>4.25</td>
<td>FCD-B Details</td>
</tr>
<tr>
<td>4.26</td>
<td>RBS Details Incorporated in FCD-B Design</td>
</tr>
<tr>
<td>4.27</td>
<td>FCD-B: Capacity Design Force Definition</td>
</tr>
<tr>
<td>4.28</td>
<td>FCD-B</td>
</tr>
<tr>
<td>4.29</td>
<td>Full-Scale FCD Test Deformed Shape, FCD-A</td>
</tr>
<tr>
<td>4.30</td>
<td>Full-Scale FCD Test Deformed Shape, FCD-B</td>
</tr>
<tr>
<td>4.31</td>
<td>Full Scale FCD Test Setup, FCD-A</td>
</tr>
<tr>
<td>4.32</td>
<td>Full Scale FCD Test Setup, FCD-B</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
</tr>
<tr>
<td>----------</td>
<td>------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>4.33</td>
<td>Welding Anchorage Rebar to Steel Precast Elements</td>
</tr>
<tr>
<td>4.34</td>
<td>Bottom and Top Wall Pours</td>
</tr>
<tr>
<td>4.35</td>
<td>FCD-A Specimen Pour</td>
</tr>
<tr>
<td>4.36</td>
<td>FCD-B Specimen Pour</td>
</tr>
<tr>
<td>4.37</td>
<td>Setup at EPM</td>
</tr>
<tr>
<td>4.38</td>
<td>Lateral Support</td>
</tr>
<tr>
<td>4.39</td>
<td>Instrumentation Map</td>
</tr>
<tr>
<td>4.40</td>
<td>Coupons Cut from FCD-A2, Built-up Section</td>
</tr>
<tr>
<td>4.41</td>
<td>Coupon Stress Strain Curves</td>
</tr>
<tr>
<td>4.42</td>
<td>FCD-A1: Clamping Device</td>
</tr>
<tr>
<td>4.43</td>
<td>FCD-B1: Clamping Device</td>
</tr>
<tr>
<td>4.44</td>
<td>FCD-A Exaggerated Rigid Body Kinematics and Free Body Diagram</td>
</tr>
<tr>
<td>4.45</td>
<td>FCD-B Exaggerated Rigid Body Kinematics and Free Body Diagram</td>
</tr>
<tr>
<td>4.46</td>
<td>FCD-A3: Deformed Shape</td>
</tr>
<tr>
<td>4.47</td>
<td>FCD Exaggerated Deformed Shape and Local Instrument</td>
</tr>
<tr>
<td>4.48</td>
<td>FCD-B3: Deformed Shape</td>
</tr>
<tr>
<td>4.49</td>
<td>FCD Exaggerated Deformed Shape</td>
</tr>
<tr>
<td>4.50</td>
<td>Contributions to FCD Deformations</td>
</tr>
<tr>
<td>4.51</td>
<td>FCD-A3: Steel End-Plate to Concrete Connection After Testing to Failure</td>
</tr>
<tr>
<td>4.52</td>
<td>FCD-A2: Working Strain Harmonic Characterization</td>
</tr>
<tr>
<td>4.53</td>
<td>FCD-A2: WSHC FCD Properties</td>
</tr>
<tr>
<td>4.54</td>
<td>FCD-A2: WSHC VE Material Properties</td>
</tr>
<tr>
<td>4.55</td>
<td>FCD-B2: Working Strain Harmonic Characterization</td>
</tr>
<tr>
<td>4.56</td>
<td>FCD-B2: WSHC FCD Properties</td>
</tr>
<tr>
<td>4.57</td>
<td>FCD-B2: WSHC VE Material Properties</td>
</tr>
<tr>
<td>4.58</td>
<td>FCD-B2: WSHC ThermaCAM Temperature Data Summary</td>
</tr>
<tr>
<td>4.59</td>
<td>FCD-A2: Higher Strain Harmonic Characterization</td>
</tr>
<tr>
<td>4.60</td>
<td>FCD-A2: HSHC FCD Properties</td>
</tr>
<tr>
<td>4.61</td>
<td>FCD-A2: HSHC VE Material Properties</td>
</tr>
<tr>
<td>4.62</td>
<td>FCD-B3: Higher Strain Harmonic Characterization</td>
</tr>
<tr>
<td>4.63</td>
<td>FCD-B3: HSHC FCD Properties</td>
</tr>
<tr>
<td>4.64</td>
<td>FCD-B3: HSHC VE Material Properties</td>
</tr>
<tr>
<td>4.65</td>
<td>FCD-B3: HSHC ThermaCAM Temperature Data Summary</td>
</tr>
<tr>
<td>4.66</td>
<td>FCD-A2: Ultimate Strain Harmonic Characterization Tests</td>
</tr>
<tr>
<td>4.67</td>
<td>FCD-A2: USHC FCD Properties</td>
</tr>
<tr>
<td>4.68</td>
<td>FCD-A2: USHC VE Material Properties</td>
</tr>
<tr>
<td>4.69</td>
<td>FCD B3: Ultimate Strain Harmonic Characterization</td>
</tr>
<tr>
<td>4.70</td>
<td>FCD-B3: USHC FCD Properties</td>
</tr>
<tr>
<td>4.71</td>
<td>FCD-B3: USHC VE Material Properties</td>
</tr>
<tr>
<td>4.72</td>
<td>FCD-B3: USHC ThermaCAM Temperature Data Summary</td>
</tr>
<tr>
<td>4.73</td>
<td>FCD-B3: 85-Storey Case Study Across-wind Results Summary</td>
</tr>
<tr>
<td>4.74</td>
<td>FCD-B3: 85-Storey Case Study Along-wind Results Summary</td>
</tr>
<tr>
<td>4.75</td>
<td>FCD-B3: 85-Storey Case Study Time Histories, 2 Times Design</td>
</tr>
<tr>
<td></td>
<td>Displacement (1/3)</td>
</tr>
<tr>
<td>4.76</td>
<td>FCD-B3: 85-Storey Case Study Time Histories, 2 Times Design</td>
</tr>
<tr>
<td></td>
<td>Displacement (2/3)</td>
</tr>
<tr>
<td>4.77</td>
<td>FCD-B3: 85-Storey Case Study Time Histories, 2 Times Design</td>
</tr>
<tr>
<td></td>
<td>Displacement (3/3)</td>
</tr>
</tbody>
</table>

**FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS**
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.78</td>
<td>FCD-B3: 85-Storey Case Study ThermaCAM Data, 2 Times Design Displacement</td>
</tr>
<tr>
<td>4.79</td>
<td>FCD-A2: Summary of Ultimate Dynamic Tests at a Frequency of 0.2 Hz for 10 Cycles</td>
</tr>
<tr>
<td>4.80</td>
<td>FCD-A3: Summary of Ultimate Dynamic Tests at a Frequency of 0.3 Hz for 5 Cycles</td>
</tr>
<tr>
<td>4.81</td>
<td>FCD-A2: End-plate Flange Connection Prior Ultimate Dynamic Test 3 (UD3)</td>
</tr>
<tr>
<td>4.82</td>
<td>FCD-A2: Crack Completely Through Web at End of Test</td>
</tr>
<tr>
<td>4.83</td>
<td>FCD-A2: Fracture Surfaces</td>
</tr>
<tr>
<td>4.84</td>
<td>FCD-A2: Removal of Specimen after Tests</td>
</tr>
<tr>
<td>4.85</td>
<td>FCD-A2: No Damage Observed in VE Material after all Tests</td>
</tr>
<tr>
<td>4.86</td>
<td>FCD-B2: Summary of Ultimate Dynamic Tests at a Frequency of 0.2 Hz for 3 Cycles</td>
</tr>
<tr>
<td>4.87</td>
<td>FCD-B3: Summary of Ultimate Dynamic Tests at a Frequency of 0.2 Hz for 3 Cycles</td>
</tr>
<tr>
<td>4.88</td>
<td>FCD-B3: White Wash After Ultimate Dynamic Test 5 (UD5)</td>
</tr>
<tr>
<td>4.89</td>
<td>FCD-B3: VE Material after Ultimate Dynamic Tests</td>
</tr>
<tr>
<td>4.90</td>
<td>FCD-A1: Ultimate Static Test with Clamping Device</td>
</tr>
<tr>
<td>4.91</td>
<td>FCD-A1: End-Plate Flange Connection Crack Formation</td>
</tr>
<tr>
<td>4.92</td>
<td>FCD-A1: Overall Deformed Shape with Clamping Device</td>
</tr>
<tr>
<td>4.93</td>
<td>FCD-A1: End-plate Flange Connection at End of Tests</td>
</tr>
<tr>
<td>4.94</td>
<td>FCD-B: Ultimate Static Tests with Clamping Device</td>
</tr>
<tr>
<td>4.95</td>
<td>FCD-B2: Clamped FCD, Prior to Ultimate Static Tests</td>
</tr>
<tr>
<td>4.96</td>
<td>FCD-B3: Ultimate Static Tests Deformed Shape</td>
</tr>
<tr>
<td>4.97</td>
<td>FCD-B2: Deformed Shape at Large Displacements</td>
</tr>
<tr>
<td>4.98</td>
<td>FCD-B2: Removal of Specimen After Tests</td>
</tr>
<tr>
<td>4.99</td>
<td>FCD-B2 Cracks at End of Test</td>
</tr>
<tr>
<td>4.100</td>
<td>Proposed FCD Ultimate Hysteresis</td>
</tr>
<tr>
<td>5.1</td>
<td>Summary of VE Material Models</td>
</tr>
<tr>
<td>5.2</td>
<td>Summary of Kelvin-Voigt Modelling Techniques</td>
</tr>
<tr>
<td>5.3</td>
<td>Constitutive Models for VE Material Behaviour Modelling</td>
</tr>
<tr>
<td>5.4</td>
<td>Implementation of Generalized Maxwell Model</td>
</tr>
<tr>
<td>5.5</td>
<td>VE Material Modelling, UT-VEM5-3: at a Frequency of 0.2 Hz and Strain of 100%</td>
</tr>
<tr>
<td>5.6</td>
<td>VE Material Modelling, UT-VEM5-3: at a Frequency of 0.2 Hz and Strain of 200%</td>
</tr>
<tr>
<td>5.7</td>
<td>VE Material Modelling, UT-VEM5-3: at a Frequency of 0.2 Hz and Strain of 400%</td>
</tr>
<tr>
<td>5.8</td>
<td>VE Material Modelling, UT-VEM5-3: at a Frequency of 0.2 Hz and Strain of 200%</td>
</tr>
<tr>
<td>5.9</td>
<td>Summary of FCD Models</td>
</tr>
<tr>
<td>5.10</td>
<td>Modelling VE FCDs with Equivalent VE Model (Model 1)</td>
</tr>
<tr>
<td>5.11</td>
<td>Modelling VE FCDs with Spring KVM (Model 2)</td>
</tr>
<tr>
<td>5.12</td>
<td>Modelling VE FCD with Generalized Maxwell Model (Model 3)</td>
</tr>
<tr>
<td>5.13</td>
<td>Velocity Sensitivity of Model 1</td>
</tr>
<tr>
<td>5.14</td>
<td>Modelling FCD-A2: WSHC3 at a Frequency of 0.5 Hz and a Targeted Strain of 50% for 500 Cycles</td>
</tr>
<tr>
<td>Figure Reference</td>
<td>Description</td>
</tr>
<tr>
<td>------------------</td>
<td>-------------</td>
</tr>
<tr>
<td>Figure 5.15</td>
<td>Modelling FCD-A2: HSHC3 at a Frequency of 0.3 Hz and a Targeted Strain of 100% for 100 Cycles</td>
</tr>
<tr>
<td>Figure 5.16</td>
<td>Modelling FCD-A2: USHC1 at a Frequency of 0.1 Hz and a Targeted Strain of 50% for 10 Cycles</td>
</tr>
<tr>
<td>Figure 5.17</td>
<td>Modelling FCD-B2: WSHC6 at a Frequency of 0.5 Hz and a Targeted Strain of 50% for 500 Cycles</td>
</tr>
<tr>
<td>Figure 5.18</td>
<td>Modelling FCD-B3: HSHC3 and a Frequency of 0.3 Hz at a Targeted Strain of 100% for 100 Cycles</td>
</tr>
<tr>
<td>Figure 5.19</td>
<td>Modelling FCD-B3: USHC1 at a Frequency of 0.1 Hz and a Targeted Strain of 200% for 10 Cycles</td>
</tr>
<tr>
<td>Figure 5.20</td>
<td>Modelling FCD-B3: AWH UL Time-History of the 85-Storey Case Study</td>
</tr>
<tr>
<td>Figure 5.21</td>
<td>Modelling FCD-B3: Landers 1992 Time-History of the 85-Storey Case Study</td>
</tr>
<tr>
<td>Figure 5.22</td>
<td>Modelling FCD-B3: Loma Prieta Time-History of the 85-Storey Case Study</td>
</tr>
<tr>
<td>Figure 5.23</td>
<td>Calibrated Model 3: HSHC3 and a Frequency of 0.3 Hz at a Targeted Strain of 100% for 100 Cycles</td>
</tr>
<tr>
<td>Figure 5.24</td>
<td>Calibrated Model 3: USHC1 at a Frequency of 0.5 Hz at a Targeted Strain of 200% for 10 Cycles</td>
</tr>
<tr>
<td>Figure 5.25</td>
<td>Calibrated Model 3: Loma Prieta Time-History of the 85-Storey Case Study</td>
</tr>
<tr>
<td>Figure 6.1</td>
<td>FCD Design Summary</td>
</tr>
<tr>
<td>Figure 6.2</td>
<td>Proposed FCD Wind Design Summary</td>
</tr>
<tr>
<td>Figure 6.3</td>
<td>Proposed Wind Design Procedure of RC Coupled Wall High-Rise Buildings with FCDs</td>
</tr>
<tr>
<td>Figure 6.4</td>
<td>Proposed FCD Seismic Design Summary</td>
</tr>
<tr>
<td>Figure 6.5</td>
<td>Proposed FCD Design Hysteresis</td>
</tr>
<tr>
<td>Figure 6.6</td>
<td>Proposed Seismic Design Procedure of RC Coupled Wall High-Rise Buildings with FCDs</td>
</tr>
<tr>
<td>Figure 6.7</td>
<td>Structural Floor Plates</td>
</tr>
<tr>
<td>Figure 6.8</td>
<td>ETABS 85-Storey Sample Structure Fundamental Mode Shapes</td>
</tr>
<tr>
<td>Figure 6.9</td>
<td>Wind Tunnel Study</td>
</tr>
<tr>
<td>Figure 6.10</td>
<td>SLS 1 in 10 year Dynamic Response as a Function of Damping</td>
</tr>
<tr>
<td>Figure 6.11</td>
<td>ULS 1 in 50 Base Moments as a Function of Damping</td>
</tr>
<tr>
<td>Figure 6.12</td>
<td>Two-FCD-A Solution: 85-Storey Building</td>
</tr>
<tr>
<td>Figure 6.13</td>
<td>Option 1: FCD-A Locations</td>
</tr>
<tr>
<td>Figure 6.14</td>
<td>Option 2: FCD-A2 Locations</td>
</tr>
<tr>
<td>Figure 6.15</td>
<td>Example of Structure with FCD Subject to SLS 1 in 10 year Wind Load</td>
</tr>
<tr>
<td>Figure 6.16</td>
<td>Dynamic Properties of Option 2 with Respect to Lintel Beam Effective Stiffness</td>
</tr>
<tr>
<td>Figure 6.17</td>
<td>Option 2: Damping Evaluation Using Logarithmic Decrement Technique</td>
</tr>
<tr>
<td>Figure 6.18</td>
<td>Governing Resultant Inter Storey Drift at Diaphragm Center of Rigidity</td>
</tr>
<tr>
<td>Figure 6.19</td>
<td>Governing Resultant Adjacent Member Design Forces (Left Line Coupling Location)</td>
</tr>
<tr>
<td>Figure 6.20</td>
<td>Governing Member Design Forces In-line with FCD (Right Line Coupling Location)</td>
</tr>
<tr>
<td>Figure 6.21</td>
<td>Landers 1992, Scaled to Match Toronto Response Spectrum</td>
</tr>
<tr>
<td>Figure 6.22</td>
<td>SLS 1 in 10 year Dynamic Response as a Function of Damping</td>
</tr>
<tr>
<td>Figure 6.23</td>
<td>ULS 1 in 50 Base Moments as a Function of Damping</td>
</tr>
</tbody>
</table>
## List of Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>TABLE 2-1</td>
<td>Examples of Effective Stiffness Values for Analysis</td>
<td>20</td>
</tr>
<tr>
<td>TABLE 2-2</td>
<td>Dynamic Response as a Function of Damping, Mass and Stiffness</td>
<td>45</td>
</tr>
<tr>
<td>TABLE 2-3</td>
<td>Case Studies</td>
<td>45</td>
</tr>
<tr>
<td>TABLE 2-4</td>
<td>Building Dynamic Properties and Motion Intensity and Duration of two 40-Storey Buildings (after Hansen et al. 1973)</td>
<td>50</td>
</tr>
<tr>
<td>TABLE 2-5</td>
<td>Noticeability of Motion Cues (after Hansen et al. 1973)</td>
<td>51</td>
</tr>
<tr>
<td>TABLE 2-6</td>
<td>Criteria for Occupant Comfort (after Chang 1973)</td>
<td>51</td>
</tr>
<tr>
<td>TABLE 4-1</td>
<td>Small-scale Loading Protocol</td>
<td>101</td>
</tr>
<tr>
<td>TABLE 4-2</td>
<td>Material Characterization Tests</td>
<td>101</td>
</tr>
<tr>
<td>TABLE 4-3</td>
<td>Wind Loading Protocol</td>
<td>104</td>
</tr>
<tr>
<td>TABLE 4-4</td>
<td>VEM5-3: Material Characterization: Results From First Cycle</td>
<td>106</td>
</tr>
<tr>
<td>TABLE 4-5</td>
<td>VEM10-1: Material Characterization: Results From First Cycle</td>
<td>107</td>
</tr>
<tr>
<td>TABLE 4-6</td>
<td>UT-VEM10-1: Material Characterization: Handheld Thermometer Temperature Measurement at Beginning and End of Tests</td>
<td>111</td>
</tr>
<tr>
<td>TABLE 4-7</td>
<td>Steel Material Properties from NSEC: JIS G 3106 SM490A</td>
<td>120</td>
</tr>
<tr>
<td>TABLE 4-8</td>
<td>High Strength Bolt Material Properties from NSEC: JIS B 1186 F10T</td>
<td>120</td>
</tr>
<tr>
<td>TABLE 4-9</td>
<td>Weld Properties from NSEC: AWS E70XX</td>
<td>120</td>
</tr>
<tr>
<td>TABLE 4-10</td>
<td>Weld Properties from Lainco: AWS E7018</td>
<td>122</td>
</tr>
<tr>
<td>TABLE 4-11</td>
<td>Mill Certificate Test Results from EL36C3J Lenton Couplers (ERICO, 2009)</td>
<td>122</td>
</tr>
<tr>
<td>TABLE 4-12</td>
<td>FCD-A Design Hierarchy of Strength</td>
<td>125</td>
</tr>
<tr>
<td>TABLE 4-13</td>
<td>FCD-B Design Hierarchy of Strength</td>
<td>129</td>
</tr>
<tr>
<td>TABLE 4-14</td>
<td>Summary of Instrumentation</td>
<td>141</td>
</tr>
<tr>
<td>TABLE 4-15</td>
<td>Tensile Coupon Results (JIS G 3106 SM490A)</td>
<td>144</td>
</tr>
<tr>
<td>TABLE 4-16</td>
<td>Tensile Coupon Results of (JIS G 3101 SS400)</td>
<td>144</td>
</tr>
<tr>
<td>TABLE 4-17</td>
<td>Concrete Cylinder Compressive Strength</td>
<td>144</td>
</tr>
<tr>
<td>TABLE 4-18</td>
<td>Full-scale Loading Protocol</td>
<td>145</td>
</tr>
<tr>
<td>TABLE 4-19</td>
<td>VE Material Strain Design Levels for Case Studies</td>
<td>146</td>
</tr>
<tr>
<td>TABLE 4-20</td>
<td>Average of WSHC Tests of FCD-A1, A2 and A-3</td>
<td>156</td>
</tr>
<tr>
<td>TABLE 4-21</td>
<td>Average of WSHC Tests of FCD-B2 and B-3</td>
<td>158</td>
</tr>
<tr>
<td>TABLE 4-22</td>
<td>Average of HSHC Tests of FCD-A1 and A2</td>
<td>161</td>
</tr>
<tr>
<td>TABLE 4-23</td>
<td>Average of HSHC Tests of FCD-B2 and B-3</td>
<td>163</td>
</tr>
<tr>
<td>TABLE 4-24</td>
<td>Average of USHC Tests of FCD-A1 and A2</td>
<td>166</td>
</tr>
<tr>
<td>TABLE 4-25</td>
<td>USHC Tests of FCD-B2</td>
<td>168</td>
</tr>
<tr>
<td>TABLE 4-26</td>
<td>FCD-A2, Summary of Ultimate Dynamic Tests</td>
<td>178</td>
</tr>
<tr>
<td>TABLE 4-27</td>
<td>FCD-A3, Summary of Ultimate Dynamic Tests</td>
<td>178</td>
</tr>
<tr>
<td>TABLE 4-28</td>
<td>FCD B-2, Summary of Tests</td>
<td>182</td>
</tr>
<tr>
<td>TABLE 4-29</td>
<td>FCD B-3, Summary of Tests</td>
<td>182</td>
</tr>
<tr>
<td>TABLE 4-30</td>
<td>ISD-111H Material Properties (NSEC 2006)</td>
<td>190</td>
</tr>
<tr>
<td>TABLE 4-31</td>
<td>Across-wind Linearized Design ISD-111H Material Properties</td>
<td>191</td>
</tr>
<tr>
<td>TABLE 4-32</td>
<td>Along-wind Linearized Design ISD-111H Material Properties</td>
<td>191</td>
</tr>
<tr>
<td>TABLE 4-33</td>
<td>Earthquake Linearized Design ISD-111H Material Properties</td>
<td>191</td>
</tr>
<tr>
<td>TABLE 4-34</td>
<td>FCD-A: Across-wind Design ISD-111H Material Properties</td>
<td>192</td>
</tr>
<tr>
<td>TABLE 4-35</td>
<td>Proposed Ultimate FCD Properties</td>
<td>193</td>
</tr>
<tr>
<td>TABLE 5-1</td>
<td>ISD-111H Material Properties (NSEC 2006)</td>
<td>204</td>
</tr>
<tr>
<td>TABLE 5-2</td>
<td>ISD-111H Material Properties (NSEC 2006) at 10% Strain</td>
<td>205</td>
</tr>
<tr>
<td>TABLE 5-3</td>
<td>GMM4TC Modelling Parameters Estimated Using Least-Squared Regression</td>
<td>205</td>
</tr>
<tr>
<td>Table Number</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>-------------</td>
<td>-----------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>TABLE 5-4</td>
<td>Comparison of Models</td>
<td>237</td>
</tr>
<tr>
<td>TABLE 6-1</td>
<td>Summary of Wind Load Cases</td>
<td>245</td>
</tr>
<tr>
<td>TABLE 6-2</td>
<td>Across-wind Linearized Design VE Material Properties</td>
<td>248</td>
</tr>
<tr>
<td>TABLE 6-3</td>
<td>Along-wind Linearized Design VE Material Properties</td>
<td>248</td>
</tr>
<tr>
<td>TABLE 6-4</td>
<td>Earthquake Material Properties</td>
<td>255</td>
</tr>
<tr>
<td>TABLE 6-5</td>
<td>Description of Load Cases</td>
<td>256</td>
</tr>
<tr>
<td>TABLE 6-6</td>
<td>Design Shear Connection Parameters</td>
<td>256</td>
</tr>
<tr>
<td>TABLE 6-7</td>
<td>Case Study Building Properties</td>
<td>261</td>
</tr>
<tr>
<td>TABLE 6-8</td>
<td>CP3 Modal Properties Assumed by Designers</td>
<td>262</td>
</tr>
<tr>
<td>TABLE 6-9</td>
<td>Two-FCD-A: SLS Across-wind Dynamic Properties for First Three Modes of Vibration</td>
<td>268</td>
</tr>
<tr>
<td>TABLE 6-10</td>
<td>One-FCD-A2: SLS Across-wind Dynamic Properties for First Three Modes of Vibration</td>
<td>269</td>
</tr>
<tr>
<td>TABLE 6-11</td>
<td>Option 1: SLS Across-wind Dynamic Properties</td>
<td>269</td>
</tr>
<tr>
<td>TABLE 6-12</td>
<td>Option 2: SLS Across-wind Dynamic Properties</td>
<td>270</td>
</tr>
<tr>
<td>TABLE 6-13</td>
<td>SLS Properties Comparison</td>
<td>270</td>
</tr>
<tr>
<td>TABLE 6-14</td>
<td>Two-FCD-A: Along-wind Design SLS,</td>
<td>274</td>
</tr>
<tr>
<td>TABLE 6-15</td>
<td>One-FCD-A2: Along-wind Design SLS</td>
<td>274</td>
</tr>
<tr>
<td>TABLE 6-16</td>
<td>Maximum Base Moment and Base Shear Comparison</td>
<td>275</td>
</tr>
<tr>
<td>TABLE 6-17</td>
<td>Two-FCD-A: ULS Adjacent Member Design</td>
<td>277</td>
</tr>
<tr>
<td>TABLE 6-18</td>
<td>One-FCD-A2 ULS Adjacent Member Design</td>
<td>277</td>
</tr>
<tr>
<td>TABLE 6-19</td>
<td>Two-FCD-A: ULS Connection Design</td>
<td>277</td>
</tr>
<tr>
<td>TABLE 6-20</td>
<td>One-FCD-A2 ULS Connection Design</td>
<td>277</td>
</tr>
<tr>
<td>TABLE 7-1</td>
<td>Comparison of Vibration Absorber and FCD System</td>
<td>284</td>
</tr>
</tbody>
</table>
LIST OF SYMBOLS

\( a_{RMax} \) maximum lateral acceleration at top of structure
\( a_{RMSmax} \) rms acceleration limit
\( a_{xi} \) modal mixing factors in the \( x \) direction for mode \( i \)
\( a_{yi} \) modal mixing factors in the \( y \) direction for mode \( i \)
\( a_{\theta i} \) modal mixing factors in the torque response for mode \( i \)
\( A \) area of bluff body surface;
\( A_{CB} \) VE material shear area
\( A_{CB}dz/dh_s \) lamina area
\( A_g \) gross beam shear area
\( A_{ve} \) effective beam shear area
\( A_w \) wall area
\( b' \) flange width at RBS
\( c \) constant for wind response;
\( C \) primary structure damping
\( (c_{0i}) \) viscous damping coefficient of Kelvin-Voigt model damper at time step \( i \) in the generalized Maxwell model
\( c_D \) equivalent viscous damping coefficient of a VE damper in a brace configuration
\( C_D \) wind drag coefficient
\( c_{FCD} \) equivalent viscous damping coefficient of a FCD damper in shear
\( C_i \) modal damping in the \( ith \) mode of vibration
\( (c_{mi}) \) viscous damping coefficient of Maxwell model damper \( m \) at time step \( i \) in the generalized Maxwell model
\( C_L \) wind lift coefficient
\( C_{pr} \) factor accounting for the effects of strain hardening and local restraint
\( C_T \) wind torque coefficient
\( c_{VE} \) viscous damping coefficient of VE material in shear in FCD
\( [C] \) damping matrix of generalized Maxwell model;
\( [D] \) damping matrix of model 2;
\( [D] \) damping matrix of model 3
\( d \) distance from extreme compression fibre to the centroid of the longitudinal reinforcement
\( D \) depth of a bluff body
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_v$</td>
<td>effective shear depth, taken as the greater of $0.9d$ and $0.72h$</td>
</tr>
<tr>
<td>$d_z$</td>
<td>lamina height</td>
</tr>
<tr>
<td>$e_x$</td>
<td>mass eccentricity in the $x$-direction</td>
</tr>
<tr>
<td>$e_y$</td>
<td>mass eccentricity in the $y$-direction</td>
</tr>
<tr>
<td>$e_R$</td>
<td>rms mass eccentricity</td>
</tr>
<tr>
<td>$E$</td>
<td>modulus of elasticity of steel</td>
</tr>
<tr>
<td>$E_c$</td>
<td>modulus of elasticity of concrete</td>
</tr>
<tr>
<td>$E_{FCD}$</td>
<td>energy dissipated per cycle of FCD</td>
</tr>
<tr>
<td>$E_{VED}$</td>
<td>energy dissipated per cycle of VE material</td>
</tr>
<tr>
<td>$E_{Si}$</td>
<td>the strain energy at the maximum displacement of the $i^{th}$ mode of vibration</td>
</tr>
<tr>
<td>$E_{VED}$</td>
<td>energy dissipated by heat in VE damper for a cycle of vibration</td>
</tr>
<tr>
<td>$E_{VEDi}$</td>
<td>energy dissipated by heat in VE damper in the $ith$ mode of vibration</td>
</tr>
<tr>
<td>$f_0$</td>
<td>frequency of excitation for a VE harmonic test</td>
</tr>
<tr>
<td>$f_1$</td>
<td>natural frequency of vibration</td>
</tr>
<tr>
<td>$f_c'$</td>
<td>specified concrete compressive strength</td>
</tr>
<tr>
<td>$f_y$</td>
<td>specified yield strength of the rebar</td>
</tr>
<tr>
<td>$F(t)$</td>
<td>harmonic load as a function of time $t$</td>
</tr>
<tr>
<td>$F_0$</td>
<td>force amplitude of harmonic load</td>
</tr>
<tr>
<td>$F_{Across}(Resonant)$</td>
<td>resonant across-wind load</td>
</tr>
<tr>
<td>$F_{Along}(Resonant)$</td>
<td>resonant along-wind load</td>
</tr>
<tr>
<td>$F_D$</td>
<td>wind drag force per unit height</td>
</tr>
<tr>
<td>$F_D$</td>
<td>mean drag component</td>
</tr>
<tr>
<td>$F_D'(t)$</td>
<td>fluctuating drag component at time $t$</td>
</tr>
<tr>
<td>$F_f$</td>
<td>1 in 50 year factored design shear force</td>
</tr>
<tr>
<td>$F_{FCDA}$</td>
<td>capacity design shear force</td>
</tr>
<tr>
<td>$F_{FCD}$</td>
<td>shear force of the FCD</td>
</tr>
<tr>
<td>$F_{FCD0}$</td>
<td>shear force amplitude of FCD</td>
</tr>
<tr>
<td>$F_{FCDA}$</td>
<td>shear force of FCD-A, calculated based on the actuator force and pin clearspan length</td>
</tr>
<tr>
<td>$F_{FCDB}$</td>
<td>shear force of FCD-B, calculated based on the actuator force and pin clearspan</td>
</tr>
<tr>
<td>$F_{FCDF}$</td>
<td>factored FCD shear force</td>
</tr>
<tr>
<td>$F_{FCDFr}$</td>
<td>probable shear force of FCD “fuse”</td>
</tr>
<tr>
<td>$F_{f(t)}$</td>
<td>base balance generalized force at time $t$ for mode $i$</td>
</tr>
<tr>
<td>$F_L$</td>
<td>wind lift force per unit height</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------------</td>
<td>-----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>$F_{\text{Max}}$</td>
<td>maximum FCD force</td>
</tr>
<tr>
<td>$F_{pr}$</td>
<td>shear force at which probable plastic moment occurs at the center of the chord length</td>
</tr>
<tr>
<td>$F_T$</td>
<td>wind torque force per unit height</td>
</tr>
<tr>
<td>$F_u$</td>
<td>ultimate strength of steel;</td>
</tr>
<tr>
<td>$F_x$</td>
<td>base shear along the $x$-axis</td>
</tr>
<tr>
<td>$F_y$</td>
<td>base shear along the $y$-axis</td>
</tr>
<tr>
<td>$F_{VE0}$</td>
<td>shear force amplitude of VE material</td>
</tr>
<tr>
<td>$\bar{F}_i$</td>
<td>mean generalized force in the $i$th mode</td>
</tr>
<tr>
<td>${F(t)}$</td>
<td>the generalized Maxwell model force vector at time $t$</td>
</tr>
<tr>
<td>$g$</td>
<td>acceleration of gravity</td>
</tr>
<tr>
<td>$G_0$,...,$G_n$</td>
<td>generalized Maxwell modelling parameters, where $n$ is the number of Maxwell models</td>
</tr>
<tr>
<td>$G_c$</td>
<td>shear modulus of concrete</td>
</tr>
<tr>
<td>$G_{c\omega}$</td>
<td>shear loss modulus of VE material</td>
</tr>
<tr>
<td>$G_E$</td>
<td>shear storage modulus of VE material</td>
</tr>
<tr>
<td>$G_{E_{\text{exp}}}(\omega_i, T_j)$</td>
<td>experimental shear storage modulus of VE material at the $i$th discrete circular excitation frequency, $\omega_i$, and the $j$th discrete temperature $T_j$</td>
</tr>
<tr>
<td>$G_S$</td>
<td>shear static storage modulus of VE material</td>
</tr>
<tr>
<td>$g_f$</td>
<td>gust response factor of a wind load</td>
</tr>
<tr>
<td>$h$</td>
<td>overall depth of beam</td>
</tr>
<tr>
<td>$h_{\text{Act}}$</td>
<td>actuator height of the base pins</td>
</tr>
<tr>
<td>$h_{\text{LVDT}}$</td>
<td>distance of the LVDT to the rotation point of the end-plate</td>
</tr>
<tr>
<td>$h_{\text{Steel}}$</td>
<td>thickness of steel layers bonding the VE material layer</td>
</tr>
<tr>
<td>$h_S$</td>
<td>storey height</td>
</tr>
<tr>
<td>$H$</td>
<td>building height</td>
</tr>
<tr>
<td>$I_e$</td>
<td>reduced effective beam inertia</td>
</tr>
<tr>
<td>$I_g$</td>
<td>gross effective beam inertia</td>
</tr>
<tr>
<td>$I_j$</td>
<td>mass moment of inertia at floor $j$</td>
</tr>
<tr>
<td>$I_{\text{CB}}$</td>
<td>coupling beam moment of inertia</td>
</tr>
<tr>
<td>$I_{\text{CB}}dz/dh_S$</td>
<td>lamina moment of inertia</td>
</tr>
<tr>
<td>$k$</td>
<td>TMD stiffness; elastic stiffness coefficient of a VE damper</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$k_0$</td>
<td>Kelvin-Voigt model elastic stiffness coefficient of generalized Maxwell model</td>
</tr>
<tr>
<td>$k_A$</td>
<td>elastic stiffness of steel FCD assembly</td>
</tr>
<tr>
<td>$k_{AA}$</td>
<td>elastic stiffness of steel FCD-A assembly</td>
</tr>
<tr>
<td>$k_{AB}$</td>
<td>elastic stiffness of steel FCD-B assembly</td>
</tr>
<tr>
<td>$k_{AL}$</td>
<td>elastic stiffness of right steel FCD assembly</td>
</tr>
<tr>
<td>$k_{AR}$</td>
<td>elastic stiffness of left steel FCD assembly</td>
</tr>
<tr>
<td>$k_B$</td>
<td>elastic stiffness of a brace</td>
</tr>
<tr>
<td>$k_D$</td>
<td>equivalent elastic stiffness coefficient of a VE damper in a brace configuration</td>
</tr>
<tr>
<td>$k_{FCD}$</td>
<td>equivalent elastic stiffness of FCD in shear</td>
</tr>
<tr>
<td>$k_{FCD_{eff}}$</td>
<td>equivalent effective elastic stiffness of FCD in shear subject to a combination of dynamic and static wind loads</td>
</tr>
<tr>
<td>$k_{FCD_{S}}$</td>
<td>static equivalent elastic stiffness of FCD in shear</td>
</tr>
<tr>
<td>$k_m$</td>
<td>Maxwell model $m$ elastic stiffness coefficient of generalized Maxwell model</td>
</tr>
<tr>
<td>$k_s$</td>
<td>slip coefficient of a bolted connection</td>
</tr>
<tr>
<td>$k_{Static}$</td>
<td>mean elastic stiffness coefficient of VE material subject to the static component of a wind load</td>
</tr>
<tr>
<td>$k_{VE}$</td>
<td>elastic stiffness coefficient of VE material</td>
</tr>
<tr>
<td>$K$</td>
<td>primary structure stiffness</td>
</tr>
<tr>
<td>$L_{CL}$</td>
<td>FCD clearspan length; pin clearspan length</td>
</tr>
<tr>
<td>$L_{CB}$</td>
<td>coupling beam length</td>
</tr>
<tr>
<td>$L_{CS}$</td>
<td>clearspan length between walls</td>
</tr>
<tr>
<td>$L_{CLA}$</td>
<td>pin clearspan length for FCD-A</td>
</tr>
<tr>
<td>$L_{CLB}$</td>
<td>pin clearspan length for FCD-B</td>
</tr>
<tr>
<td>$L_{RBS}$</td>
<td>length between RBS sections</td>
</tr>
<tr>
<td>$L_W$</td>
<td>length of wall</td>
</tr>
<tr>
<td>$m$</td>
<td>TMD mass</td>
</tr>
<tr>
<td>$m_j$</td>
<td>mass at storey $j$</td>
</tr>
<tr>
<td>$m_{Storey}$</td>
<td>storey mass</td>
</tr>
<tr>
<td>$M$</td>
<td>bending moment; primary structure mass</td>
</tr>
<tr>
<td>$M_a$</td>
<td>the moment caused by the axial forces in the connecting beams</td>
</tr>
<tr>
<td>$M_i$</td>
<td>bending moment of wall $i$; generalized mass of the $i$th mode</td>
</tr>
<tr>
<td>$M_{Max}$</td>
<td>maximum FCD bending moment</td>
</tr>
<tr>
<td>$M_p$</td>
<td>plastic bending resistance of built-up FCD steel sections</td>
</tr>
<tr>
<td>$M_{pr}$</td>
<td>probable bending resistance of built-up FCD steel sections</td>
</tr>
</tbody>
</table>
List of Symbols

- $M_{pCL}$: plastic bending resistance of built-up FCD steel sections at the face of the wall
- $M_{pRBS}$: plastic bending resistance of built-up FCD steel sections at RBS
- $M_{prRBS}$: probable bending resistance of built-up FCD steel sections at RBS
- $M_x$: base moment of high-rise building about the $x$-axis
- $M_x(t)$: base balance moment in the $x$-axis at a time $t$
- $M_y$: yield bending resistance of built-up FCD steel sections;
- $M_y(t)$: base moment of high-rise building about the $y$-axis
- $M_y(t)$: base balance moment about the $y$-axis at a time $t$
- $M_{yAnc}$: yield bending resistance of FCD anchorage
- $M_{yCL}$: yield bending resistance of built-up FCD at the face of the wall
- $M_{yEP}$: yield bending resistance of built-up FCD end-plate
- $M_{yRBS}$: yield bending moment at built-up RBS
- $M_z$: base moment of high-rise building about the $z$-axis
- $M_z(t)$: base balance moment about the $z$-axis at a time $t$
- $[M]$: generalized Maxwell model mass matrix
- $n$: layers of VE material;
- $N$: number of cycles used to calculate logarithmic decrement
- $N_Cycles$: number of cycles
- $N_{Steel}$: number of steel layers
- $N_{VE}$: number of VE layers
- $NL$: coupling moment of two walls
- $N_i L_i$: coupling moment of wall $i$
- $p$: temperature shifting function parameter used for generalized Maxwell model;
- $P_u$: axial force in a wall;
- $P_{Act}$: number of cycles used to calculate logarithmic decrement
- $P_{Channel}$: the axial force at the base of the wall resulting from earthquake loading combinations
- $P_u$: sum of the actuator force
- $q$: axial channel force
- $q_z$: shear flow at height $z$
- $r$: RBS cut radius
- $r'$: fluctuating component of gust in the $z$ direction
- $r_z$: linear slope of increasing damping with amplitude of vibration
- $r_z$: total response component including base moments, deflections, accelerations, torsional velocity or other loads subject to a wind storm;
- $R$: radius of circular TSD tank
- $R$: resistance of structural member in series with FCD “fuse”
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R_{Max}$</td>
<td>structural peak response of the dynamic response component including base moments, deflections, accelerations, torsional velocity or other loads subject to a wind storm</td>
</tr>
<tr>
<td>$R_{MaxAlong}$</td>
<td>maximum structural peak response in the along-wind direction including base moments, deflections, accelerations, torsional velocity or other loads subject to a wind storm</td>
</tr>
<tr>
<td>$R_{MaxAcross}$</td>
<td>maximum structural peak response in the across-wind direction including base moments, deflections, accelerations, torsional velocity or other loads subject to a wind storm</td>
</tr>
<tr>
<td>$R_yF_y$</td>
<td>expected yield stress of RBS steel</td>
</tr>
<tr>
<td>$\bar{R}$</td>
<td>quasi-static mean structural peak response for the duration of the storm base moments and deflections or other loads subject to a wind storm</td>
</tr>
<tr>
<td>$s$</td>
<td>RBS chord length; specific heat of VE material</td>
</tr>
<tr>
<td>$S_p(f)$</td>
<td>wind force spectral density at a frequency $f$</td>
</tr>
<tr>
<td>$S_g(f)$</td>
<td>wind gust spectral density at a frequency $f$</td>
</tr>
<tr>
<td>$S_d(f)$</td>
<td>wind displacement response spectral density at a frequency $f$</td>
</tr>
<tr>
<td>$t$</td>
<td>time</td>
</tr>
<tr>
<td>$t_i$</td>
<td>time step $i$</td>
</tr>
<tr>
<td>$t_w$</td>
<td>reinforced concrete wall thickness</td>
</tr>
<tr>
<td>$T_i$</td>
<td>period of wind signal; VE material temperature</td>
</tr>
<tr>
<td>$T_0$</td>
<td>VE material temperature at the $ith$ time step</td>
</tr>
<tr>
<td>$T_1$</td>
<td>ambient temperature at beginning of test</td>
</tr>
<tr>
<td>$T_{ref}$</td>
<td>structure natural period of vibration</td>
</tr>
<tr>
<td>$T_{ref}$</td>
<td>reference temperature used for generalized Maxwell model</td>
</tr>
<tr>
<td>$T_{VE}$</td>
<td>VE material temperature</td>
</tr>
<tr>
<td>$T_{VE0}$</td>
<td>VE material temperature at beginning of tests</td>
</tr>
<tr>
<td>$T_{VEEnd}$</td>
<td>VE material temperature at end of loading</td>
</tr>
<tr>
<td>$T_{VELB}$</td>
<td>lower bound design VE material temperature</td>
</tr>
<tr>
<td>$T_{VEUB}$</td>
<td>upper bound design VE material temperature</td>
</tr>
<tr>
<td>$T(t)$</td>
<td>base balance torque at a time $t$</td>
</tr>
<tr>
<td>$u$</td>
<td>top storey displacement</td>
</tr>
<tr>
<td>${u(t)}$</td>
<td>generalized Maxwell model displacement vector at time $t$</td>
</tr>
<tr>
<td>$\bar{u}$</td>
<td>mean static deflection</td>
</tr>
<tr>
<td>$u_0$</td>
<td>harmonic displacement amplitude of VE material</td>
</tr>
<tr>
<td>$u_A$</td>
<td>built-up steel assembly displacements relative to FCD walls</td>
</tr>
<tr>
<td>$u_{AL}$</td>
<td>left built-up steel assembly displacement relative to wall of FCD</td>
</tr>
<tr>
<td>$u_{AR}$</td>
<td>right built-up steel assembly displacement relative to wall of FCD</td>
</tr>
</tbody>
</table>
List of Symbols

$u_b$ built-up steel assembly bending deformations of FCD

$u_{bL}$ left built-up steel assembly bending deformations of FCD

$u_{bR}$ right built-up steel assembly bending deformations of FCD

$u_{Dynamic}$ dynamic VE shear displacement in an along-wind load

$u_{EP}$ shear deformation caused by end-plate rotation

$u_{FCD}$ FCD shear displacement

$u_{FCD0}$ FCD shear displacement amplitude

$u_{FCDMax}$ maximum FCD shear displacement

$u_{Max}$ maximum FCD shear displacement

$u_{Mid}$ The average mid span FCD displacement at center of VE material relative to left and right walls

$u_{MidL}$ The average mid span FCD displacement at center of VE material relative to left wall

$u_{MidR}$ The average mid span FCD displacement at center of VE material relative to right wall

$(u_n)_i$ free vibration of peak amplitude $n$ of for the $ith$ mode of vibration and

$u_{Static}$ static VE shear displacement in an along-wind load

$u_v$ total built-up steel assembly shear deformation of FCD

$u_{vL}$ left built-up steel assembly shear deformation of FCD

$u_{vR}$ right built-up steel assembly shear deformation of FCD

$u_{VE}$ VE material displacement

$u_{VEMax}$ maximum VE material shear displacement

$u_{VEpr}$ VE displacement at the probable FCD shear force when “fuse” is activated

$\ddot{u}_{Dynamic}$ dynamic VE material shear displacement in an along-wind load

$\{\ddot{u}(t)\}$ generalized Maxwell model velocity vector at time $t$

$\ddot{u}_{Across}$ across-wind acceleration at top storey

$\ddot{u}_{Along}$ along-wind acceleration at top storey

$\{\ddot{u}(t)\}$ generalized Maxwell model acceleration vector at time $t$

$\bar{u}_{xj}$ mean static deflection in $x$ -direction of storey $j$

$\bar{u}_{yj}$ mean static deflection in $y$ -direction of storey $j$

$\bar{u}_{\theta j}$ mean static deflection in torque of storey $j$

$u_{xj}^*$ condensed diaphragm displacement in $x$ -direction of storey $j$

$u_{yj}^*$ condensed diaphragm displacement in $y$ -direction of storey $j$

$u_{\theta j}^*$ condensed diaphragm displacement in $\theta$ -direction of storey $j$

$v$ ‘cycling rate’ or effective frequency of the response used in gust factor calculation

$v'$ fluctuating component of gust in the $x$ direction
List of Symbols

$V_f$  factored shear force

$V_r$  FCD shear force at which 5% probability of slip occurs

$V_{r,max}$  maximum shear capacity of a beam cross-section

$V_x(t)$  base balance shear force in the $x$ direction at a time $t$

$V_y(t)$  base balance shear force in the $y$ direction at a time $t$

$\bar{V}(z)$  mean wind speed at height $z$

$\bar{V}_{10}$  mean wind velocity at height 10$m$

$\bar{V}_g$  gradient mean wind speed

$w$  uniform lateral line load

$W$  dissipated energy per volume of VE material

$w'$  fluctuating component of gust in the $y$ direction

$x$  number of discrete frequencies used for nonlinear parameter estimation

$x_1, x_2, x_3$  Newmark-β constants

$x_4, x_5, x_6$  number of discrete temperatures used for nonlinear parameter estimation

$Z_{E}$  effective plastic modulus of the beam section at the reduced beam section

$z_g$  gradient height of wind

$\alpha$  power law coefficient for wind

$\alpha_{EP}$  total FCD end-plate rotation

$\alpha_{EPL}$  left FCD end-plate rotation

$\alpha_{EPR}$  right FCD end-plate rotation

$\alpha_w$  concrete section property reduction factor used for wall effective stiffness properties

$\alpha_T(T, T_{ref})$  temperature shifting function which adjusts the generalized Maxwell modelling parameters from a reference temperature $T_{ref}$ to $T$

$\beta$  Newmark-β constant;

$\beta_0$  constant used for temperature calibration with the generalized Maxwell model

$\beta_{0,ref}$  modelling parameter used for the generalized Maxwell models at a reference temperature

$\Gamma_{VE}$  VE material stiffness factor for equivalent VE stiffness method

$\gamma$  Newmark-β constant

$\gamma_c$  concrete density

$\gamma_0$  strain amplitude of VE material in a harmonic cycle

$\bar{\gamma}$  quasi-static mean strain of VE material in a wind storm

$\gamma_{Max}$  maximum shear strain of VE material

$\gamma_{Static}$  static shear strain of VE material in the along-wind direction
List of Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_S$</td>
<td>static shear strain of VE material</td>
</tr>
<tr>
<td>$\gamma_{VE1}$</td>
<td>VE material shear strain for Option 1</td>
</tr>
<tr>
<td>$\gamma_{VE2}$</td>
<td>VE material shear strain for Option 2</td>
</tr>
<tr>
<td>$\gamma_{VED}$</td>
<td>dynamic shear strain of VE material</td>
</tr>
<tr>
<td>$\gamma_{VEM}$</td>
<td>maximum shear strain of VE material</td>
</tr>
<tr>
<td>$\gamma_{VEM}$</td>
<td>mean shear strain of VE material in the along-wind direction</td>
</tr>
<tr>
<td>$\gamma(t)$</td>
<td>shear strain of VE material</td>
</tr>
<tr>
<td>$\gamma_{VE}$</td>
<td>VE material strain</td>
</tr>
<tr>
<td>$\gamma_{VE0}$</td>
<td>VE material strain amplitude in a harmonic cycle</td>
</tr>
<tr>
<td>$\gamma(t)$</td>
<td>shear strain rate of VE material</td>
</tr>
<tr>
<td>$\delta_A$</td>
<td>relative vertical displacement at ends of lamina caused by axial deformations</td>
</tr>
<tr>
<td>$\delta_i$</td>
<td>logarithmic decrement in the $ith$ mode of vibration</td>
</tr>
<tr>
<td>$\delta_M$</td>
<td>relative vertical displacement at ends of lamina caused by bending deformations</td>
</tr>
<tr>
<td>$\delta_V$</td>
<td>relative vertical displacement at ends of lamina caused by shear deformations</td>
</tr>
<tr>
<td>$\delta_\theta$</td>
<td>relative vertical displacement at ends of lamina caused by rotations of wall cross sections about their centerlines</td>
</tr>
<tr>
<td>$\Delta$</td>
<td>change of length of string potentiometers</td>
</tr>
<tr>
<td>$\Delta_{EPL}$</td>
<td>uplift of left FCD end-plate</td>
</tr>
<tr>
<td>$\Delta_{EPR}$</td>
<td>uplift of right FCD end-plate</td>
</tr>
<tr>
<td>${ (\Delta F)_i }$</td>
<td>generalized Maxwell model incremental force vector at time step $i$;</td>
</tr>
<tr>
<td>$[ (\Delta \ddot{F})_i ]$</td>
<td>model 2 incremental force vector at time $t$;</td>
</tr>
<tr>
<td>$\Delta F_{ECD}$</td>
<td>model 2 incremental effective load vector at time step $i$;</td>
</tr>
<tr>
<td>$\Delta F_{ECD}^*$</td>
<td>model 2 incremental condensed effective FCD force at time step $i$;</td>
</tr>
<tr>
<td>$\Delta F_{VE}$</td>
<td>generalized Maxwell model incremental VE force at time step $i$</td>
</tr>
<tr>
<td>$\Delta F_{VE}^*$</td>
<td>generalized Maxwell model incremental condensed effective force at time step $i$</td>
</tr>
<tr>
<td>$\Delta_{S1F}$</td>
<td>change of length of front string potentiometer number 1</td>
</tr>
<tr>
<td>$\Delta_{S2F}$</td>
<td>change of length of front string potentiometer number 2</td>
</tr>
<tr>
<td>$\Delta_{S3F}$</td>
<td>change of length of front string potentiometer number 3</td>
</tr>
<tr>
<td>$\Delta_{S4F}$</td>
<td>change of length of front string potentiometer number 4</td>
</tr>
<tr>
<td>$\Delta_{S5F}$</td>
<td>change of length of front string potentiometer number 5</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$\Delta S_{1R}$</td>
<td>change of length of rear string potentiometer number 1</td>
</tr>
<tr>
<td>$\Delta S_{2R}$</td>
<td>change of length of rear string potentiometer number 2</td>
</tr>
<tr>
<td>$\Delta S_{3R}$</td>
<td>change of length of rear string potentiometer number 3</td>
</tr>
<tr>
<td>$\Delta S_{4R}$</td>
<td>change of length of rear string potentiometer number 4</td>
</tr>
<tr>
<td>$\Delta S_{5R}$</td>
<td>change of length of rear string potentiometer number 5</td>
</tr>
<tr>
<td>$\Delta T$</td>
<td>change in temperature of VE material caused by dissipation of energy as heat</td>
</tr>
<tr>
<td>${(\Delta u)_i}$</td>
<td>model 2 incremental displacement vector at time step $i$; model 3 incremental displacement vector at time step $i$</td>
</tr>
<tr>
<td>${(\Delta \dot{u})_i}$</td>
<td>model 2 incremental velocity vector at time step $i$; model 3 incremental velocity vector at time step $i$</td>
</tr>
<tr>
<td>$(\Delta u_{FCD})_i$</td>
<td>model 3 incremental FCD displacement at time step $i$; generalized Maxwell model incremental damper displacement at time step $i$</td>
</tr>
<tr>
<td>$(\Delta \dot{u}_{FCD})_i$</td>
<td>model 2 incremental FCD velocity at time step $i$; model 3 incremental FCD velocity at time step $i$</td>
</tr>
<tr>
<td>$(\Delta u_{VE})_i$</td>
<td>model 2 incremental damper displacement at time step $i$; model 3 incremental damper displacement at time step $i$</td>
</tr>
<tr>
<td>$(\Delta \dot{u}_{VE})_i$</td>
<td>model 2 incremental damper velocity at time step $i$; model 3 incremental damper velocity at time step $i$</td>
</tr>
<tr>
<td>$\eta$</td>
<td>loss factor of VE material</td>
</tr>
<tr>
<td>$\eta_D$</td>
<td>equivalent VE loss factor of brace configuration damper</td>
</tr>
<tr>
<td>$\eta_{exp}(\omega_i, T_j)$</td>
<td>experimental loss factor of VE material at the $i$th discrete circular excitation frequency, $\omega_i$, and the $j$th discrete temperature $T_j$</td>
</tr>
<tr>
<td>$\eta_{FCD}$</td>
<td>equivalent VE loss factor of FCD in shear</td>
</tr>
<tr>
<td>$\eta_{VE}$</td>
<td>loss factor of VE material for equivalent VE method</td>
</tr>
<tr>
<td>$\theta$</td>
<td>VE material phase angle</td>
</tr>
<tr>
<td>$\theta_{FCD}$</td>
<td>FCD phase angle in shear</td>
</tr>
<tr>
<td>$\theta_{VE}$</td>
<td>VE material phase angle in FCD</td>
</tr>
<tr>
<td>$\kappa$</td>
<td>thermal conductivity of the VE material</td>
</tr>
<tr>
<td>$\Lambda$</td>
<td>non-dimensional temperature correction factor</td>
</tr>
<tr>
<td>$\mu$</td>
<td>mass ratio of TMD and primary structure</td>
</tr>
<tr>
<td>$\nu$</td>
<td>cycling rate or effective frequency of the response</td>
</tr>
<tr>
<td>$\xi$</td>
<td>equivalent damping ratio of the viscoelastic damper</td>
</tr>
<tr>
<td>$\xi_0$</td>
<td>damping at the low amplitude plateau</td>
</tr>
<tr>
<td>$\xi_1$</td>
<td>damping rise during the linear increasing amplitude range</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$\xi_{\text{Added}}$</td>
<td>added damping from FCD to high-rise structure</td>
</tr>
<tr>
<td>$\xi_{\text{eff}}$</td>
<td>effective damping of a damped primary structure with TMD</td>
</tr>
<tr>
<td>$\xi_{\text{eqi}}$</td>
<td>equivalent damping in the $i$th mode of vibration of system with added VE dampers; equivalent damping in the $i$th mode of vibration of system with added FCDs</td>
</tr>
<tr>
<td>$\xi_i$</td>
<td>modal damping of the $i$th mode of vibration</td>
</tr>
<tr>
<td>$\xi_{\text{Inherent}}$</td>
<td>inherent damping of high-rise structure</td>
</tr>
<tr>
<td>$\xi_{\text{max}}$</td>
<td>damping threshold value</td>
</tr>
<tr>
<td>$\xi_s$</td>
<td>primary structure damping</td>
</tr>
<tr>
<td>$\xi_{\text{Total}}$</td>
<td>total damping of high-rise structures as a sum of added damping of FCDs and inherent damping</td>
</tr>
<tr>
<td>$\rho$</td>
<td>density of air;</td>
</tr>
<tr>
<td>$\sigma_R$</td>
<td>mass density of VE material</td>
</tr>
<tr>
<td>$\sigma_{\text{PT}}$</td>
<td>root mean square of the dynamic response component including base moments, deflections, accelerations, torsional velocity or other loads</td>
</tr>
<tr>
<td>$\sigma_v^2$</td>
<td>axial wall post-tensioning stress of full-scale tests</td>
</tr>
<tr>
<td>$\sigma_F^2$</td>
<td>variance of the wind signal</td>
</tr>
<tr>
<td>$\sigma_u^2$</td>
<td>variance of the force</td>
</tr>
<tr>
<td>$\sigma_{u_j}^2$</td>
<td>variance of the displacement response in the $x$ direction at floor $j$</td>
</tr>
<tr>
<td>$\sigma_{u_j}^2$</td>
<td>variance of the displacement response in the $y$ direction at floor $j$</td>
</tr>
<tr>
<td>$\sigma_{u_{\theta_j}}^2$</td>
<td>variance of the displacement response in torque at floor $j$</td>
</tr>
<tr>
<td>$\sigma_{u_j}^2$</td>
<td>variance of the displacement response at floor $j$</td>
</tr>
<tr>
<td>$\sigma_{u_{\theta}}^2$</td>
<td>variance of the torsional velocity</td>
</tr>
<tr>
<td>$\sigma_u^2$</td>
<td>variance of the acceleration</td>
</tr>
<tr>
<td>$\sigma_{u_j}^2$</td>
<td>variance of the acceleration in the $x$ direction at floor $j$</td>
</tr>
<tr>
<td>$\sigma_{u_{\theta_j}}^2$</td>
<td>variance of the acceleration in the $y$ direction at floor $j$</td>
</tr>
<tr>
<td>$\sigma_{u_{\theta_j}}^2$</td>
<td>variance of the acceleration in torsion at floor $j$</td>
</tr>
<tr>
<td>$\sigma_B^2$</td>
<td>variance of the background response</td>
</tr>
<tr>
<td>$\sigma_R^2$</td>
<td>variance of the resonant response</td>
</tr>
<tr>
<td>$\sigma_M^2$</td>
<td>variance of the resonant base moment</td>
</tr>
<tr>
<td>$\sigma_{M_x}^2$</td>
<td>variance of the resonant base moment in the $x$ direction</td>
</tr>
<tr>
<td>$\sigma_{M_y}^2$</td>
<td>variance of the resonant base moment in the $y$ direction</td>
</tr>
<tr>
<td>$\sigma_T^2$</td>
<td>variance of the resonant base torque</td>
</tr>
<tr>
<td>$\sigma_\gamma$</td>
<td>root mean squared of VE material strain</td>
</tr>
<tr>
<td>$\sigma_{\gamma_{\text{Along}}}$</td>
<td>root mean squared of VE material strain in the along-wind direction</td>
</tr>
</tbody>
</table>
List of Symbols

\( \sigma_{\gamma \text{Across}} \) \quad \text{root mean squared of VE material strain in the across-wind direction}
\( \tau(t) \) \quad \text{shear stress of VE material at time } t
\( \tau_E(t) \) \quad \text{elastic shear stress of the material at time } t
\( \tau_C(t) \) \quad \text{viscous shear stress of the material at time } t
\( \tau_{E0} \) \quad \text{stress in Kelvin-Voigt dashpot used for generalized Maxwell model}
\( \tau_m \) \quad \text{stress in Maxwell dashpot for element } m \text{ for generalized Maxwell model}
\( \lambda \) \quad \text{cross-sectional shape factor in shear}
\( \phi \) \quad \text{load resistance factor}
\( \phi_c \) \quad \text{resistance factor for concrete (0.65)}
\( \phi_{xij} \) \quad \text{mode shape } i \text{ in the } x \text{ direction on the floor } j
\( \phi_{yij} \) \quad \text{mode shape } i \text{ in the } y \text{ direction on the floor } j
\( \phi_{bij} \) \quad \text{mode shape } i \text{ in torque on the floor } j
\( \phi_{x}(z_j) \) \quad \text{mode shape } i \text{ in the } x \text{ direction at height } z
\( \phi_{y}(z_j) \) \quad \text{mode shape } i \text{ in the } y \text{ direction at height } z
\( \phi_{b}(z_j) \) \quad \text{mode shape } i \text{ in torque at height } z
\( \{ \phi_i \} \) \quad \text{mode shape } i
\( \psi_1 \ldots \psi_n \) \quad \text{modelling parameter used for the generalized Maxwell models, where } n \text{ is the number of Maxwell models}
\( \psi_m \) \quad \text{modelling parameter used for the generalized Maxwell model, maxwell element } m
\( \psi_{1, \text{ref}} \ldots \psi_{n, \text{ref}} \) \quad \text{modelling parameter used for the generalized Maxwell models at a reference temperature, where } n \text{ is the number of Maxwell models}
\( \psi_{m, \text{ref}} \) \quad \text{modelling parameter used for the generalized Maxwell model, maxwell element } m \text{ at a reference temperature}
\( \omega \) \quad \text{circular excitation frequency}
\( \omega_0 \) \quad \text{circular excitation frequency at harmonic frequency } f_0
\( \bar{\omega} \) \quad \text{circular excitation frequency of vibration absorber harmonic load}
\( \omega_a \) \quad \text{circular frequency of attached vibration absorber}
\( \omega_i \) \quad \text{circular frequency of the } i\text{th mode of vibration}
\( \omega_n \) \quad \text{circular frequency of TLCD}
\( \omega_s \) \quad \text{circular frequency of primary structure}
CHAPTER 1: INTRODUCTION

1.1 BACKGROUND AND MOTIVATION

Urbanization in Canada and across the world has resulted in a dramatic increase in the demand for construction of tall buildings. The increase in the number of high-rise structures in major urban centers is the result of reaching the practical limits of urban sprawl coupled with an increased cost of land in major cities worldwide, including Canadian cities such as Vancouver and Toronto. In Toronto, high-rise condominium sales represent the majority of new housing sales (Ladurantaye 2010). In growing cities in the Middle East and Asia, the trend towards high-rise construction is even more striking; tall buildings in these areas of the world account for 70% of new building construction. Approximately 1000 buildings have been constructed worldwide since 2001 between 31-40 stories, and 700 between 41-60 stories (Emporis 2010), accounting for an estimated market of $20 billion dollars per year. The number of supertall buildings (over 80 stories) constructed has increased dramatically as well, especially in China.

As buildings increase in height and slenderness they become increasingly sensitive to lateral dynamic vibrations. Lateral wind vibrations increase the lateral inertial loads on the building structure and when lateral accelerations and torsional velocities exceed certain levels, the vibrations may be perceived by building occupants. This can cause various levels of occupant discomfort varying from minor annoyance to extreme motion sickness. For high-rise structures it is very common for the Service Limit State (SLS), which includes motion perception criteria, to be the most important design aspect and to govern the design of the lateral load resisting system of the structure.

When structural engineers are faced with building occupant motion perception problems, there are only a limited number of methods available to solve them: i) reducing the number of stories, ii) stiffening the lateral load resisting system, iii) changing the structural layout, iv) incorporating a vibration absorber within the building or v) installing supplemental damping elements throughout the structure.

The structural response can be mitigated by reducing the number of stories, which lowers the natural period of vibration and decreases the surface area over which wind loading is applied. However, decreasing the number of stories in the building can jeopardize the economic viability of the project, because the higher stories are the most profitable units due to their high market price relative to the increase in overall...
cost that their construction represents.

Stiffening the structure is typically accomplished by increasing the structural member sizes or the material strength. This increases the cost of the building materials, while reducing the rentable space of the units and still may not suffice to reduce the structural vibrations significantly.

Structural engineers can also change the structural layout of the building. This is usually accomplished by adding walls, outriggers or a completely new lateral load resisting system layout. Altering the floor plan affects the architectural layout which will affect the functionality and the rentable space of the building. Generally the structural design is first carried out and then an expensive wind tunnel study is conducted. After the results are received from the wind tunnel consultants, modifications must be made to the structural system in order to address any problems that have arisen during the wind tunnel study. By the time this is done, the design of the building and the unit layouts may be nearly finalized. Any changes made in the structural layout can require significant changes to the design services which may be quite advanced at this stage of the project.

If the above methods are not viable solutions, supplemental damping devices can be added to the structure. The most common method is to add vibration absorbers, which are large tuned-mass or tuned-liquid dampers (TMDs or TLDs). Vibration absorbers are both complex and expensive to design and are mainly used in the construction of supertall buildings. Typically they are located at the top floor of the building, which is the most efficient location to mitigate wind vibrations, because this is where the largest lateral displacements occur in the first mode of vibration. Unfortunately, this is also the most valuable real estate. The TMDs and TLDs are comprised of a large amount of additional mass, typically 2-4% of the total building mass, which results in large gravity loads which must be carried over the height of the building. Also, the vibration absorber must be tuned to one specific frequency which is sometimes hard to define at the design stage, can change over time, and needs to be measured and monitored and adjusted in place in order for the vibration absorber to be effective over the life of the building. If the vibration absorber is tuned to an incorrect frequency it may be largely ineffectively and may actually amplify structural motions. For wind applications they are tuned to the structure's predominant natural frequency and therefore may not be beneficial for higher mode vibrations, which are common in high-rise buildings subject to earthquakes. TMDs and TLDs have been shown to be ineffective for seismic loads and are often designed to lock in-place in the event of an earthquake. The vibration absorber then merely acts as an added mass at the top of the building.

In the past, supplemental passive viscous and viscoelastic dampers have also been used for mitigating lateral wind and earthquake vibrations of buildings. These dampers have mainly been used in low to mid-rise buildings, but there have been some high-rise applications where numerous dampers have been distributed throughout the height of the structure typically in axial brace configurations. The kinematics of
commonly constructed reinforced concrete buildings, though, do not produce enough axial deformations in these brace type elements for the configuration to be effective in mitigating lateral vibrations. A benefit of distributed supplemental passive damping systems is that they can add damping to many modes of vibration and therefore will mitigate both wind and earthquake vibrations. Another benefit is that the performance of these distributed dampers is less sensitive to the uncertainty of building properties compared to vibration absorbers.

Damping is essentially the measure of energy dissipation in a vibrating system, being the absorption of the kinetic energy that is imparted to the structure by external loading. The inherent damping properties of high-rise buildings, compared to the building mass and stiffness, are very difficult to determine precisely. Full-scale measurements in high-rise buildings are difficult to obtain and there are no generalized methods for calculating a high-rise structure’s inherent damping properties. Also, the existing field measurements have shown a large degree of scatter, decreasing with the height of the building, often lower than those assumed at the design stage.

Distributed linear viscous damping is very effective in reducing both wind and earthquake vibrations. Analytical models of tall buildings subject to wind vibrations have shown that as little as an additional 2% equivalent viscous damping will have a substantial impact on the dynamic response of high-rise structures. In certain cases, an additional 2% equivalent viscous damping can reduce the wind-induced acceleration and velocity levels by as much as 40%.

The most common lateral load resisting system for residential high-rise apartment and condominium buildings in Canada are reinforced concrete (RC) coupled wall systems (which are commonly referred to as coupled shear wall systems). It is also the most common form of high-rise residential construction in major growth areas such as China and the Middle East (Fang 1991, Emporis 2010). It is an extremely efficient laterally load resisting system for buildings from 30 to 60 stories tall. Figures 1.1a and 1.1b show photos of coupled wall buildings under construction in Seattle and in New York City. The system consists of large concrete walls, which typically divide individual residential units, coupled together with lintel beams over the wall perforations, as illustrated in Figure 1.1c. The lintel beams provide a shear connection between the walls, causing axial forces in the walls when subjected to lateral deformations, substantially increasing the lateral stiffness of the system compared to two uncoupled cantilevered walls. As a result of the coupling between the two walls, the lintel beams undergo large shear loads and shear displacements under lateral wind and earthquake loading. The large relative deformations in the concrete lintel beams can be seen in Figure 1.1d where a schematic of the kinematics of a coupled wall building subject to lateral loading is shown.
Further to this, high-rise concrete coupled wall buildings are also designed for buildings in high-seismic regions. When extreme earthquake events occur, coupling beams and the concrete walls at their base are designed to yield and act as “fuse” elements. The nonlinear response of these elements is intended to form a global ductile mechanism. All other elements in the structure are designed to remain essentially linear elastic when the ductile yielding elements are undergoing their large nonlinear deformations. This is assured using capacity design principles. The addition of distributed damping also helps to reduce earthquake vibrations.

1.2 OBJECTIVE OF WORK

The previous discussion suggests that damping elements configured strategically throughout a coupled wall high-rise building, in locations with large deformations under lateral deformations, could have a significant effect on the dynamic response of the structure.

A simple and effective device that can add distributed linear structural damping to a system when effectively configured, is a viscoelastic (VE) damper acting in shear. When VE damping material is excited by dynamic loads in shear, there is a combination of an elastic and viscous response. For a given cycle of deformation, the VE material will return to its original position with some energy lost as heat that is radiated into the surrounding environment. The force-displacement relationship of a VE damper under harmonic excitation is shown in Figure 1.2a. The energy dissipated in a cycle of motion is the area enclosed by this force-displacement loop.
As discussed earlier, in coupled wall buildings, lintel beams are locations that undergo large shear deformations. Coupling beams would therefore be an effective location for VE dampers activated in shear, because they will be able to dissipate large amounts of energy in a vibration cycle. This rationale lead to the development of the VE Fork Configuration Damper (FCD). Figure 1.2b, shows the FCD introduced in lieu of a typical coupling beam connecting two walls. A space is left in-between the slab and the FCD so that the FCD can deform without interference. The FCD consists of multiple layers of VE material sandwiched between and bonded to multiple steel plates. The VE material alternates between layers of steel plates with each consecutive steel plate extending out to the opposite side. For the FCD shown in Figure 1.2c all steel plates bonded to the VE material are bolted together in a slip-critical connection with built-up I-sections and filler plates. The built-up I-sections are welded onto steel end-plates which have welded couplers for rebar to be threaded into, anchoring the FCD into the walls.

The multiple steel plates form two fork shaped elements, with the forks connected by the layers of VE material. The steel plate-VE assembly consists of \( n \) layers of VE material sandwiched in between \( n + 1 \) steel plates. The FCD allows for the \( n \) layers of VE material to act in parallel when subjected to shear deformations.

The storey view of an FCD configured in an actual RC coupled wall building application is illustrated in Figure 1.3a. When the building is subjected to lateral wind excitations, the walls deflect laterally and large rotations are introduced into the walls, causing relative displacements at both ends of the FCD producing large shear deformations in the VE material layers. The deformed shape of two walls coupled together with FCDs is illustrated in Figure 1.3b. The deformed shape of the FCD and a storey view of the building are shown in Figures 1.3c and 1.3d.
CHAPTER 1: Introduction

FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

Under an extreme event, such as a rare earthquake, the FCD will be subjected to large shear displacements at both wall anchorages. In order to ensure there is no tearing of the VE material itself or large forces introduced into the walls from the VE deformation, a “fuse” element is built into the FCD that activates at a predefined load. The performance of the FCD is ensured through a capacity design procedure.

The objective of this study was to develop and validate the FCD device for use in high-rise buildings for enhanced performance during wind and earthquake vibrations by adding distributed linear viscous damping through strategic placement of the FCD system. This must be done economically and non-obtrusively. The introduction of the FCDs must be able to reduce the dynamic response of the structure with minimal increases to the interstorey drifts or the natural periods of vibration.

Figure 1.3 Kinematics of Buildings Equipped with FCDs
1.3 SCOPE OF WORK

In Chapter 2, current high-rise building design and the behaviour of buildings subject to lateral loading is discussed. The chapter focuses on the dynamic behaviour of RC coupled wall buildings, the current design strategies and procedures for wind and seismic loading and the current methods to mitigate lateral vibrations. Chapter 3 introduces the Fork Configuration Damper (FCD) and describes the evolution of the FCD design through a collaboration between engineers at the structural firm Halcrow Yolles and professors at the University of Toronto. Many configurations and design details are discussed as alternatives to produce a constructible and economic FCD. Chapter 4 describes the experimental program that was conducted on the FCD to validate its use in high-rise buildings. There were two sets of tests conducted on 11 samples provided in-kind by Nippon Steel Engineering (NSEC): i) five small-scale VE damper tests to validate the hysteretic VE material properties conducted at the University of Toronto (UofT) Structural Testing Facilities and ii) six full-scale tests of two types of FCDs designed for actual high-rise buildings conducted at the Structures Laboratory at École Polytechnique in Montreal (EPM). Chapter 5 describes analytical techniques which are used to model VE material behaviour and their application to modelling full-scale FCDs. These techniques are used to model the FCD under realistic wind and earthquake vibrations. Simplified modelling techniques are then validated for use in 3-Dimensional Finite Element (FE) software to determine the damping and overall performance in high-rise buildings. Chapter 6 describes proposed design procedures for high-rise buildings followed by a case study that investigates the inclusion of FCDs in an actual high-rise building in downtown Toronto. Finally in Chapter 7, a summary of the FCD system and conclusions are presented along with recommendations for future studies.
CHAPTER 2: BEHAVIOUR AND DESIGN OF RC COUPLED WALL HIGH-RISE BUILDINGS

2.1 INTRODUCTION

The analysis and design of high-rise building lateral load resisting systems is a challenging and complex topic. Typical high-rise buildings constructed today are more slender, lighter and more flexible than high-rise buildings constructed in the past, which relied on heavy and large structural members. As a result of this current tall buildings are more dynamically sensitive to wind and seismic excitations. For these reasons, the lateral design of these structures has become increasingly challenging and more evolved than the gravity design of such structures.

In wind storms, lateral loads and vibrations are introduced into high-rise buildings from the lateral wind pressure striking the large surface area of the building facade, dynamically exciting the building in its first few natural modes of vibration. In seismic events, ground accelerations and displacements introduce building displacements and dynamic inertial loads caused by the dynamic amplification of the ground motion.

In practice, high-rise buildings are designed to remain essentially linear and undamaged, except for some minor cracking, for all gravity and 1 in 10 year Service Limit State (SLS) and 1 in 50 year Ultimate Limit State (ULS) wind loads. The current wind design strategy is to make the lateral load resisting system as stiff and strong as possible while satisfying architectural constraints. For seismic design, pre-selected ductile fuse elements are distributed throughout the structure, providing stable, concentrated energy dissipation without critical strength deterioration for the Design Level Earthquake (DLE). For reinforced concrete (RC) coupled wall structures, the ductile fuse elements are structural walls at the base of the structure and lintel beams coupling the walls throughout the height of the structure, specially detailed for earthquakes. It is intended that damage be concentrated only in these areas. Properly detailed RC structural wall systems have performed well during earthquakes and are considered effective for ensuring life safety (Fintel 1991, Paulay and Priestley 1992 and Atimtay and Recaep 2006).
This chapter aims to introduce the main design objectives and performance of coupled wall high-rise buildings. First, RC coupled wall buildings are introduced, followed by a description of their inherent dynamic properties. Next, current wind and seismic design philosophies are introduced along with lateral design procedures. Lastly, some of the methods commonly used to mitigate lateral vibrations are discussed.

2.2 INTRODUCTION TO RC COUPLED WALL BUILDINGS

Coupled wall buildings consist of two or more walls (commonly called shear or structural walls) in the same plane connected by lintel beams. Coupled walls provide an efficient means of gravity and lateral load resistance while providing openings between the walls that are essential for building functionality.

It is also very common for the elevator cores to be surrounded by structural walls, that become part of the coupled wall system. This adds to the horizontal stiffness of the structure, but because elevator cores are typically located near the center of rigidity they contribute little to the torsional resistance compared to external tube systems. External tube systems have structural members which are located at large distances from the building plan center of rigidity. For this reason, wind loads may cause torsional velocity and acceleration problems for coupled wall buildings, especially when there is coupling of the torsional and translational modes of vibration.

Coupled walls are commonly used in high-rise apartment and condominium buildings. They serve the needs of residential construction, because the walls can separate adjacent units and the coupling beams can easily be hidden over corridors or door openings. Architects and owners favour this system, because of the flexible unit division, acoustic and fire separation between units and large unobstructed window views compared to other lateral load resisting systems. For these reasons coupled walls have been widely adopted as the most common type of residential high-rise construction worldwide.

2.2.1 Modelling of RC Coupled Wall Buildings

Coupled walls with shear resistant coupling lintel beams, act as composite cantilevers rotating about the common centroidal axis of the coupled walls in their plane. If the coupling beams are infinitely rigid, the system will behave as a single very large composite cantilever. The axial stress caused by the applied moment on the composite cantilever varies linearly with the maximum compressive stress and maximum tension stress occurring at the extreme edges. The stress varies linearly across the composite elements, as illustrated in Figure 2.1a. When the connecting beams between the two walls are infinitely flexible, the composite unit will behave as two independent cantilevers, with the stress varying as illustrated in Figure 2.1b. In general the behavior will be somewhere between these two bounds. Figure 2.1c graphically illustrates the superposition of stresses of coupled walls subject to lateral loads (Coull and Choudry 1967 and Smith and Coull 1991). The composite cantilever stiffness is substantially larger than the two uncoupled cantilever walls bending about their respective axes.
When the walls deflect due to the applied lateral loads, the coupling beams rotate and displace vertically bending in double curvature. Due to this action, shear forces are introduced into the lintel beams, which cause bending moments in the opposite direction to the external applied moments on each wall. The beam shear forces also induce tensile forces into the windward wall and compressive forces into the leeward wall. The deformed shape of coupled walls subject to lateral loads is illustrated in Figure 2.2b. The resultant total storey moment, $M$, is expressed as:

$$
M = \sum_{i=1}^{n} M_i + \sum_{i=1}^{n} N_i L_i
$$  \hspace{1cm} (2.1)

where $n$ is the number of walls, $M_i$ is the moment in each individual wall, and $N_i L_i$ is the coupling moment about the centerline of the composite section which is the result of the axial force in each wall, $N_i$, multiplied by the distance between the centroid of each wall to the centroid of the composite section, $L_i$. If there are only two walls connected by a single beam, Eq. (2.1) reduces to:
where $L$ is the distance between the centroidal axes of the two walls. Figure 2.2c illustrates the free body diagram of each wall for this case. When the coupling beams are infinitely rigid, the term $\sum_{i=1}^{\infty} N_i L_i$ is maximum, and when the coupling beams are infinitely flexible, the term vanishes.

**Figure 2.2 Coupled Wall Subject to Lateral Wind Loads**

**2.2.1.1 Continuum Approach**

In the 1960’s Beck (1962), Coull and Choudhury (1967), Rosman (1960, 1964 and 1966) and others provided the basis for understanding the kinematic response and flow of forces in coupled wall structures with the development of the continuous medium method. This method was the most commonly used analysis method, prior to the advent of commercially available FE modelling software.
The approach consists of replacing the connecting beams in a coupled wall building shown in Figures 2.3a and 2.3b with a continuous connection, in the form of a lamina of height $dz$. The lamina has an area $A_{CB} dz/h_s$ and a moment of inertia $I_{CB} dz/h_s$, which is distributed along the height of the building, as shown in Figures 2.3b, 2.3c and 2.3e. $I_{CB}$ is the coupling beam moment of inertia and $A_{CB}$ is the coupling beam area. Due to the fact that the structural walls are much stiffer than the beams, points of contraflexure occur at the midspan of the beams and similarly at the midspan of the continuous lamina, which is rigidly connected to the wall. If the continuous connections are cut along their centerline, then the forces acting along the cut lamina are represented by a shear flow of intensity, $q(z)$, per unit height and axial force of intensity, $n(z)$, per unit height, as shown in Figure 2.3c. The axial force $N$ in each wall at any height $z$ can then be determined by taking the integral of the shear flow $q$ in the connecting medium above that height:

$$N = \int_{z}^{H} q \, dz$$  \hspace{1cm} (2.3)$$

Differentiating Eq. (2.3): 

$$q = -\frac{dN}{dz}$$  \hspace{1cm} (2.4)$$
Assuming that the base of the structure is fixed, vertical compatibility occurs at the cut ends of the cantilevered lamina caused by i) wall rotations, ii) wall axial coupling, iii) bending and iv) shear deformations in the beams as shown in Figures 2.4a, 2.4b, 2.4c and 2.4d. All vertical compatibility elements are described below:

i) The relative vertical displacement, $\delta_0$, is at the ends of the lamina caused by rotations of the wall cross sections about their centerline, $dx/dz$, due to wall bending moments. This bending moment comes from two sources: the free bending of the walls due to the applied external forces and the bending in the opposite direction caused by the shear forces and axial forces in the connecting lamina. The relative displacement, $\delta_0$, at height $z$ is:

$$\delta_0 = (L_1/2 + L_{CB}/2)(dx/dz) + (L_2/2 + L_{CB}/2)(dx/dz) = L(dx/dz) \quad (2.5)$$

where $L_1$ and $L_2$ are the lengths of walls 1 and 2, $L_{CB}$ is the length of the coupling beam and $L$ is the distance between the centerlines of the two walls.

ii) The relative vertical displacement, $\delta_A$, is caused by axial deformations of the walls under the axial loads, $N$. The shear forces from the lamina will induce tensile forces in the windward wall and compressive forces in the leeward wall. The relative displacement, $\delta_A$, at height $z$ is:

$$\delta_A = -\frac{1}{E_c} \frac{\int_0^z Ndz}{(1/A_1 + 1/A_2)} \quad (2.6)$$

iii) The relative vertical displacement, $\delta_M$, is caused by the bending deformations of the lamina of depth, $dz$, cantilevered from the end of the wall. The cantilever has a flexural rigidity $(E_c(I_{CB}/h_S))dz$ and subject to a tip force, $qdz$. The relative displacement caused by bending deformations, $\delta_M$, at height $z$ is:

$$\delta_M = -\frac{qdz}{3E_c(I_{CB}/h_S)dz}(L_{CB}/2)^3 = \frac{-qh_S L_{CB}^3}{12E_c I_{CB}} \quad (2.7)$$

iv) The relative vertical displacement, $\delta_V$, is caused by shear deformations of the lamina of depth, $dz$, cantilevered from the end of the wall, having a shear rigidity $(G_s(A_{CB}/\lambda h_S))dz$, subject to a tip force, $qdz$. The relative displacement caused by shear deformations, $\delta_V$, at height $z$ is:

$$\delta_V = -\frac{qdz}{(G_s(A_{CB}/\lambda h_S))dz}(L_{CB}/2) = \frac{-qh_S L_{CB}}{G_s A_{CB} \lambda} \quad (2.8)$$

where $\lambda$ is a cross-sectional shape factor in shear ($\lambda = 1.2$ for rectangular beams).

There is no relative displacement at the point of contraflexure of the connecting beams, therefore to ensure vertical compatibility:

$$\delta_0 + \delta_M + \delta_V + \delta_A = 0 \quad (2.9)$$
In Eq. (2.9) the relative displacements $\delta_M$, $\delta_V$ and $\delta_A$ are in the opposite direction of $\delta_0$. Considering free bending due to the externally applied moment, $M$, and reverse bending caused by the shears and axial forces in the connecting medium as seen in Figure 2.3c, the moment curvature relationship for both walls at any level is:

$$EI_1\left(\frac{d^2x}{dz^2}\right) = M_1 = M - \left(L_1/2 + L_{CB}/2\right) \int_0^H qdz - M_a$$  \hspace{1cm} (2.10)$$

$$EI_2\left(\frac{d^2x}{dz^2}\right) = M_2 = -\left(L_2/2 + L_{CB}/2\right) \int_0^H qdz + M_a$$  \hspace{1cm} (2.11)$$

where $M_a$ is the moment caused by the axial forces in the connecting beams. The sum of Eq. (2.10) and Eq. (2.11) yields the overall moment-curvature relationship of the coupled walls:

$$E(I_1 + I_2)\left(\frac{d^2x}{dz^2}\right) = \int_0^H qdz = M - LN$$  \hspace{1cm} (2.12)$$

The solution to this is equation is provided by many references for different load cases (Beck 1962, Smith and Coull 1991 and Rosman 1960, 1964 and 1966). The results of the solution give a great deal of information of the performance of the system with different coupling beam and wall stiffnesses and geometries.
2.2.1.2 Example of the Continuum Approach

To demonstrate coupled wall behavior, a 40-storey RC structure, height of $H = 120\text{m}$, with two planar coupled walls in parallel is investigated. The concrete compressive strength is $f'_c = 50\text{MPa}$, with a concrete modulus of $E_c = 33 \times 10^3 \text{MPa}$, and the lateral load is an assumed uniform lateral line load of $w = 30\text{kN/m}$. Each of the planar coupled walls are comprised of two walls, each with a length of $L_1 = L_2 = 7\text{m}$ connected by coupling beams with length $L_{CB} = 2\text{m}$ over the height of the building. The wall thicknesses are $500\text{mm}$ and four different lintel beam depths are investigated: 1) slab alone $h_{CB} = 200\text{mm}$ of width $2000\text{mm}$, 2) $h_{CB} = 350\text{mm}$ including a $200\text{mm}$ slab of width $2000\text{mm}$, 3) $h_{CB} = 550\text{mm}$ including a $200\text{mm}$ slab of width $2000\text{mm}$ and 4) $h_{CB} = 750\text{mm}$ including a $200\text{mm}$ slab of width $2000\text{mm}$. The storey deflection, beam shear flow, wall axial load and wall bending moments, ($M_1 + M_2$), are illustrated for the various beam depths in Figures 2.5a, 2.5b, 2.5c and 2.5d, respectively.

![Figure 2.5 RC 40-storey Coupled Wall Structure Subject to Uniform Load](image-url)
When the walls are coupled with just the slab, the deflection is similar to two independent cantilevers whereas, when the coupling beam depth is \(750\,mm\), the deflection is similar to a composite cantilever. The shear forces in the beams are very small at the base of the structure, where it is not deforming much, and reach a maximum between about 1/5 of the building height for the stiffest beams and about 2/5 of the building height for just the slab. The axial wall forces are caused by the coupling beam shear forces, and reach a maximum at the base of the structure. The coupled wall structure with the deepest beams has the largest axial forces in the walls due to the largest degree of coupling. The sum of the wall moments \(M = M_1 + M_2\) are maximum at the base of the structure. Near the top of the structure the sum of the wall moments actually acts in the opposite direction to the moments at the base of the structure, due to the beam coupling forces and relatively small moment arm available to generate moment due to the applied lateral load.

### 2.2.1.3 Finite Element Approach

Currently, 3-dimensional finite element models guide the analysis of complex high-rise RC structure; some of the important assumptions and techniques for the analysis of buildings are outlined below. The stiffness of the floors in the horizontal direction is much larger than that of the walls, so each floor is modelled as a rigid diaphragm causing each floor slab to move in its horizontal plane as a rigid body (Smith and Coull 1991 and Ghali and Neville 1997). Generally the bending and shear contributions to lateral load resistance provided by the slabs are neglected in the lateral analysis. Horizontal forces and masses are assumed to act at each floor level, lumping the lateral forces and moments at the center of rigidity and the masses at the center of mass. Using static condensation, the 3-dimensional MDOF structure is transformed into 3 dynamic degrees of freedom per storey (two lateral directions and one torsional) significantly reducing the complexity of the analysis. Each degree of freedom along the floor slab of the uncondensed structure is related by geometry to condensed degrees of freedom. P-delta effects are included in the analysis as they can significantly increase the building deflections. The contributions to the lateral load resistance of the structure by gravity columns are sometimes ignored in these models unless an outrigger system connects the shear walls connecting the core to the gravity columns.

For relatively simple structures, without geometric irregularities, a plane frame approach could be used to analyze the response in each direction, as illustrated in Figures 2.6a and 2.6c. In this approach, the walls and coupling beams are both modelled as frame elements. The coupling beam frame elements span between the centerlines of both walls with rigid offsets at both ends with effective lengths that are typically taken as the length of the wall divided by 2. For the walls and beams, shear deformations can be significant and should be taken into account in the analysis. Lateral loads are assumed to be lumped at the center of rigidity of each floor. The wall frame elements bend and the rigid offsets connecting the centerline of the wall to the beams rotate which causes large relative deformations between the ends of the beam elements.
CHAPTER 2: Behaviour and Design of RC Coupled Wall High-Rise Buildings

For complex structures, full 3-D finite element models are used to model and analyze the behavior of coupled wall structures. An example of a complex floor plan shape is illustrated in Figure 2.6d. In this case, the center of rigidity, which is based on wall geometry and stiffness distribution, is slightly offset from the geometric center of the slab. The center of mass is located further from the center of the core than the center of rigidity. This causes the lateral and torsional mode shapes to be coupled, which can increase the wind-induced response of the structure. Also, dynamic lateral loads at the center of mass are offset from the center of rigidity causing additional dynamic torsional excitations. Closed cores centered around the center of rigidity, with walls far from the center of rigidity are preferred, because such systems provide large...
lever arms and shear flow continuity to resist the torsion. Unfortunately, the functional layout of the structure often limits the torsional stiffness, because the elevator core layout is dictated by the elevator locations and tenants prefer to have minimal walls in their apartments blocking views.

To generate a more economical structure, generally the wall and coupling beam member sizes and concrete compressive strengths decrease with height at distinct levels. To avoid large stress concentrations at a single level, changing both member sizes and concrete strength simultaneously is avoided.

Current 3-D finite element modelling software packages use shell elements to model the walls. The gross effective wall properties are modified to account for cracking in the in-plane directions and the out-of-plane stiffnesses are assumed to be small and not included in the analysis. When adjacent walls are stacked together forming a core, they become more stable in the out-of-plane direction. Frame elements are used to model the coupling beams spanning between wall shell element nodes located at the faces of openings. An elevation view of an coupled wall structure modelled using shell elements and frame elements is illustrated in Figure 2.6b.

Ideally, walls carrying large gravity loads over the height of the structure are continued to the foundation, however architects and owners often desire open spaces at ground levels, so large transfer slabs or transfer girders are used to carry the gravity loads to external columns and then to the foundations.

Figure 2.7 illustrates the deflected shape of a 3-D coupled wall building modelled using ETABS (CSI 2005), subject to lateral loading in Figure 2.7a and torsional loading in Figure 2.7b. Storey views in Figure 2.7c show the large deformations in the coupling beams in the direction of action of the load. The storey view of the rotational deflected shape in Figure 2.7d shows the large shear deformations concentrated in the core wall coupling beams, providing shear flow continuity with the core walls in their own plane.

### 2.2.1.4 Effective Stiffness Properties Used for Modelling

The Design of Concrete Structures, CSA A23.3-04, clause 8.6.2.2 (CSA 2004) states that in lieu of test results of similar concrete elements, the modulus of elasticity, $E_c$, with a density, $\gamma_c$, between $1,500 < \gamma_c < 2,500 \text{kg/m}^3$ may be taken as:

$$E_c = (3,300 \sqrt{f'_c} + 6,900)(\gamma_c/2300)^{1.5}$$

(2.13)

where $f'_c$ is the specified compressive strength of the concrete.

The CSA A23.3-04 (CSA 2004) allows for some flexibility in the way designers account for the amount and distribution of cracking in concrete structures. For ease of analysis, the gross effective moment of inertia and gross area, $(I_g, A_g)$, are reduced to the effective moment of inertia and area, $(I_e, A_e)$. The effective properties are used for the analysis at an expected design level of stress or strain.
Clause 10.14.1.2 gives some general guidelines on how to account for the cracking of concrete elements and Clause 21.2.5.2.1 gives guidelines for cracking during elastic seismic analyses. Paulay and Priestley (1992) give estimates of member stiffness for analyses where shear deformations are accounted for with modified moment of inertia values.

Iterative procedures are sometimes used by structural engineering design firms to determine a more realistic stiffness distribution of the coupling beams over the height of the building. The procedure starts with an initial estimate of the effective values for the SLS design parameters and then uses the code-based...
or wind tunnel-based design loads to determine the distribution of forces throughout the coupling beams. After the beam force distribution is determined, the stiffness coefficients of the members that are heavily stressed are reduced and the stiffness coefficients of the members that are lightly stressed are increased accordingly. Further iterations are performed until the calculation of forces in the members are within 10% of the previous iteration. The same distribution of cracking parameters are used in the ULS analysis as well, but adjusted to represent more cracking. Table 2-1 outlines the analysis parameters along with common initial estimates.

TABLE 2-1  Examples of Effective Stiffness Values for Analysis

<table>
<thead>
<tr>
<th>Clause 21.2.5.2.1</th>
<th>Paulay and Priestley (1992)</th>
<th>Common SLS Wind First Guess</th>
<th>Common ULS Wind First Estimates</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_e$ $A_{ve}$</td>
<td>$I_e$ $A_{ve}$</td>
<td>$I_e$ $A_{ve}$</td>
<td>$I_e$ $A_{ve}$</td>
</tr>
<tr>
<td>Non-diagonally Reinforced Coupling Beams</td>
<td>$0.4I_g$ $0.15A_g$</td>
<td>$0.2I_g \left[ 1 + 3\left( \frac{h}{L_{CB}} \right)^2 \right]$</td>
<td>$0.4I_g$ $0.4A_g$</td>
</tr>
<tr>
<td>Diagonally Reinforced Coupling Beams</td>
<td>$0.25I_g$ $0.45A_g$</td>
<td>$0.4I_g \left[ 1 + 3\left( \frac{h}{L_{CB}} \right)^2 \right]$</td>
<td>$0.4I_g$ $0.4I_g$</td>
</tr>
<tr>
<td>Cracked Walls</td>
<td>$\alpha_w I_g$ $\alpha_w A_g$</td>
<td>$\left( \frac{100}{f'_y} + \frac{P_u}{f'_c A_g} \right)I_g$</td>
<td>$0.9I_g$ $A_g$</td>
</tr>
<tr>
<td>Foundation Walls</td>
<td>NA NA</td>
<td>NA</td>
<td>$0.5I_g$ $A_g$</td>
</tr>
</tbody>
</table>

In the table, $h$ is the depth of the lintel beam, $L_{CB}$ is the coupling beam clearspan, $\alpha_w = 0.6 + P_u / (f'_c A_g) \leq 1.0$ and $P_u$ is the axial force at the base of the wall resulting from earthquake loading combinations.

Unfortunately, the methods of determining the properties of the system often do not translate well to high-rise building design and therefore the corresponding design-based deflections and accelerations could be in error. Actual dynamic properties measured in full-scale, high-rise structures are further discussed in Section 2.2.3.
2.2.1.5 Examples of FE Modelling Uncertainties

There are many uncertainties in the modelling of RC coupled wall structures, and some of the uncertainties are outlined below:

- **Concrete Cracks** develop with time in many concrete beams and walls in the building. To account for this, engineers use reduced gross effective properties based upon an expected design limit state event. In reality, cracking varies over the life of the structure and varies based on the specific type of loading and direction. Also, the cracking is not uniform over the height of the structure; engineers sometimes perform an iterative analysis attributing more cracking to the higher stressed members redistributing the loads.

- **Stiffness contribution from rebar** in the walls and beams that varies from typical design values are ignored, but for heavily or lightly reinforced lintel beams this may be significant (Priestley 2003).

- **Axial elongation** effects due to shear cracking of coupling beams is difficult to account for with current software and is not well understood.

- **Concrete modulus of elasticity** under dynamic loads is often higher than the static values used.

- **Soil-structure interaction** is typically ignored in the analyses.

- **Non-structural elements** such as partitions, cladding and stairways and even structural elements such as the slab, out-of-plane walls and gravity columns are ignored, but they may contribute significantly to the stiffness of the structure.

- **Structural restraint at the base** is generally assumed in FE models, but there are many locations along the basement levels to the ground floor that may offer restraint that is not accounted for (Isyumov et al. 2010).

2.2.2 Inherent Damping Properties of RC Buildings

Damping is a measure of how much energy is dissipated when the structure deforms dynamically and how quickly a vibrating structure comes to rest as a result of this damping effect. Damping plays a particularly significant role in controlling the vibrations of structures for frequencies near resonance. The amount of damping depends on member stress and excitation levels, construction materials and structural configurations. It is also one of the most difficult building parameters to quantify. Currently methods to estimate damping are based on full-scale measurements of built structures. There is large uncertainty in estimating the damping properties of high-rise buildings and there are no clear guidelines, although recently there has been an increased research interest in better characterizing and quantifying damping in high-rise buildings because predicted dynamic vibrations are extremely sensitive to the damping parameters assumed in the modelling of modern high-rise buildings.
When a building is vibrating in mode $i$, the damping ratio, $\xi_i$, is a quantification of the energy that is dissipated per cycle in that mode of vibration. This value is specified as a percentage of the critical value of damping, which is the energy dissipation required in a mode of vibration that would damp out a free vibration of the system in a single cycle. In actual building structures the concept of an equivalent viscous damping value was introduced mainly for its mathematical simplicity, which assumes a viscous mechanism that does not necessarily exist in typical buildings.

In the wind loading commentary of the National Building Code of Canada, NBCC 2005 (NRC 2006) it is suggested that the inherent damping values assumed for analyses must be based mainly on experiments of real structures, and the value commonly used for concrete frames is 2%. Currently, for the design of high-rise buildings, structural engineers generally assume equivalent damping ratios of 1.5% to 2% of critical for the SLS analysis and design and 2% to 2.5% of critical for the ULS analysis for all modes of vibration.

The work of many researchers on the damping of high-rise buildings determined that inherent damping was extremely variable and highly dependent on the amplitude of vibration (Hart 1996, Jeary 1986, Jeary 1996 and Kareem and Gurley 1996). Other parameters that influenced inherent damping were the height of the building, natural frequency, structural dimensions, type of material used, geological site conditions and non-structural building elements.

Damping in buildings consists of two sources of energy dissipation: i) building material damping at low vibration amplitudes, ii) friction of non-structural elements and energy dissipated in the main structural elements with increasing amplitude caused either by friction mechanisms in connections (an example being bolt slippage in bolted steel structures) or by the onset of low-level nonlinearities (such as minor cracking in concrete elements). Hysteretic yielding of structural members mobilized during extreme events such as earthquakes is also sometimes referred to as damping, however it is not seen as a viable mechanism for mitigating wind vibrations because it causes permanent damage and deformations to the structure. This damage is unacceptable for frequent wind events.

Jeary described three different response ranges for the damping of buildings for increasing amplitudes, which are illustrated in Figure 2.8 (Jeary 1986 and Jeary 1996):

- **Low Amplitude Plateau** - At very low amplitude vibrations, large structural elements undergo the majority of the differential movement. This region is governed by the structural system. Using Jeary’s model the damping at the low amplitude plateau is termed $\xi_0$ and is correlated with the fundamental frequency of the building.
• **Increasing Amplitude Range** - As the amplitude of vibration increases, energy is dissipated through material micro-cracking. This region involves a nonlinear damping characteristic and is governed by the material type. For this region, the damping increases at a rate of $r_\xi$ with increasing amplitude of vibration until a threshold value of $\xi_{\text{max}}$ is reached. For high-rise buildings there is evidence that this plateau is smaller than for low-rise RC structures.

• **High-Amplitude Plateau** - In this region the amplitude of motion is large enough such that all damping mechanisms excluding hysteretic yielding of structural elements are mobilized and remain constant. Galambos and Mayes (1979), showed that this high amplitude plateau may be associated with a reduction in damping due to smoothening of rubbing surfaces.

After the high-amplitude plateau, the main structural members begin to yield and dissipate energy. For coupled wall buildings the majority of energy dissipation takes place in the lintel beams throughout the height of the structure, and in the walls at the base.

Tamura (2006) introduced the concept of a “critical tip deflection”, which is the amplitude at the top of the building when damping is at the maximum value. Beyond the maximum damping value there is structural yielding at much larger amplitudes. Tamura suggested that the maximum damping occurs at low tip deflections, on the order $20 \times 10^{-6}$ of the height of the building.

A recent study by Bentz and Kijewski-Correa (2008) hypothesized that the amount of damping in a building depends on the relative participation of the structural system’s dominant deformation mechanisms: Frame-racking (moment frames) versus cantilever action (coupled shear walls). The preliminary findings indicated that damping decreased with increased cantilever action which was predicted by modes shapes using FE modelling.
2.2.3 Measured Damping and Frequency Values of High-Rise Buildings

There are four main techniques to obtain damping of structures in the field: i) the vibration decay method, ii) half-power bandwidth method, iii) the autocorrelation method and iv) the random vibration method. Each of these analysis techniques has advantages and disadvantages, but it is important to recognize that damping values obtained are often influenced by the vibration-testing method (Davenport and Hill-Carroll 1986, Kareem and Gurley 1996 and Tamura and Suganuma 1996). Bendat and Piersol (2000) summarized the techniques well which are used to obtain the damping ratios and natural frequencies using random data.

Frequency and damping measurements from 205 real buildings in Japan, as reported by Satake et al. (2003), Suda et al. (1996) and Tamura et al. (2000) and 185 buildings in Asia, Europe and North America, as reported by Lagomarsino (1993) have been compiled. The studies from Japan were conducted on 137 steel-framed, 25 reinforced concrete (RC) and 43 mixed steel-framed reinforced concrete (SRC) buildings. The majority of data on the RC/SRC buildings were collected for buildings ranging from 50 to 100 meters tall and most data was collected for small vibration amplitudes, making it difficult to extrapolate rules for the damping and stiffness of high-rise buildings subject to larger vibration amplitudes. The testing methods included forced vibrations from mechanical shakers, pull and release vibrations, as well as vibrations induced by wind and seismic events. The data gives general trends for buildings based on height and frequency, but actual structure-specific data is withheld. Figures 2.9a, b and c show the general trends of the dynamic properties of 205 buildings (Satake 2003) and Figure 2.9d shows an amplitude dependent damping response of a tall concrete building and will be discussed later (Li et al. 2003). This data exhibited significant scatter, but the following conclusions were reached:

- The damping ratio in the first mode of vibration reduced for increasing height of the structure and increasing the natural period. This can be seen in Figures 2.9a and 2.9b.
- The natural periods of vibration exhibited less scatter in comparison to the damping values as seen in Figures 2.9b and 2.9c.
- As expected the natural periods of vibration increased with the height as illustrated in Figure 2.9c. This data was fitted to a linear equation and a predictor for the natural period of vibration \( T_1 = H/67 \) was derived, where \( H \) is the height in meters.
- The periods of vibration measured were approximately 80% of those used for design.
- For buildings of heights ranging from 10 m to 100 m the damping of concrete buildings was expressed as: \( \xi_1(\%) = 1.43f_1 + 4.7(u/H) - 0.18 \), where \( f_1 \) is the natural frequency, \( u \) is the top storey displacement amplitude in meters and \( (u/H < 2 \times 10^{-5}) \) is the non-dimensional amplitude.
- Damping values in structures with pile foundations were larger than those on spread foundations.
• Hotels and apartments exhibited somewhat higher values of damping than office buildings. This was thought to be due to the additional partition walls in hotels and apartments.

• In comparing the two main orthogonal directions of buildings, more damping was observed for vibrations along the larger dimensions, particularly in office buildings.

• Torsional damping was slightly less than translational damping.

• Higher mode damping was approximately 40% greater than lower mode damping.

• For RC buildings 100 m to 130 m tall the average damping value was 1% with the highest value at 1.8% and the lowest at 0.6%.

Lagomarsimo (1993) described 185 buildings ranging from 45 m in height to 330 m in height, made of steel, RC and mixed construction. The data again exhibited substantial scatter, but similar conclusions were reached, the clearest being that damping decreased with height and frequency and that the natural frequency decreased with height. Lagomarsimo gave predictors for the natural period of vibration as

\[ T_1 = \frac{H}{55} \]

where \( H \) is the height of the building in meters, a value 22% higher than suggested by Satake.

Jeary (1986) investigated 16 buildings in the U.K. under low amplitude vibrations. There was little information on the structural properties of the buildings and there were not many high-rise buildings included in the study. The results came up with predictors for the natural period of vibration and damping for buildings in terms of height, \( T_1 = \frac{H}{46} \) and \( \xi_1 = \frac{92}{H} \) for the high amplitude plateau as defined by Jeary (Jeary and Ellis 1984, Ellis 1996 and Jeary 1996).

The Chicago full-scale monitoring project is the first of its kind where wind experts from the University of Notre Dame and the University of Western Ontario and structural engineers from Skidmore, Owens and Merril LLP (SOM) and Samsung Engineering & Construction have collaborated to validate analytical and scaled models through full-scale monitoring of three high-rise structures in Chicago (Kilpatrick et al. 2003 and Kijewski-Correa et al. 2006). Two of these buildings are steel framed tube structures and one is a reinforced concrete building with shear walls near the core of the building and two outriggers tying the core to the perimeter columns. All three buildings are office type construction, have significant separation between the translational and torsional modes and are rectangular in plan. No information was given on the height of the buildings, to protect their identity. The buildings were monitored during magnitude 4.5 and 5.0 earthquake events and two very significant wind events including a “mild hurricane” that caused damage to some buildings in the area. The first three periods of vibration of the concrete building were 10 to 25% less than predicted by FE software. This was attributed to the in-situ concrete elements not reaching their assumed service life cracked properties or to the in-situ modulus of elasticity being greater than assumed in the FE analysis. The measured damping values of 1.42% and 2.4% in the translational directions and 3.59% in the torsional direction were higher than predicted. The first of the two steel buildings exhibited natural frequencies close to the predicted values with a damping of 0.87%.
The second had natural periods of vibration that were 10-15% higher than predicted, with damping values ranging from 1.04 to 1.33%.

Celebi (1996) published a study on the dynamic properties of five buildings under low-amplitude vibrations before and after the 1989 Loma Prieta, California earthquake. The Transamerica Building in San Francisco (60-storey pyramidal steel moment frame) and the Pacific Park Plaza in Emeryville (30-storey RC moment resisting frame) were two of these structures. The Transamerica Building exhibited similar properties before and after the Loma Prieta earthquake. The natural frequencies were about 0.34 Hz in both translational directions with damping of 0.8% in one direction and 1.4% in the other direction during low amplitude excitations before and after the Loma Prieta earthquake. Measurements from the accelerometers during the earthquake showed that the natural frequencies fell to 0.28 Hz while the damping increased to 2.2% and 4.9% in both translational directions. Measurements of the natural frequencies in both translational directions of the Pacific Park Plaza made before the earthquake were 0.59 Hz and after dropped to 0.48 Hz. Measurements during the earthquake showed that the natural frequency was 0.38 Hz. Prior to the Loma Prieta earthquake, the damping values were 2.6% in both translational directions and after the earthquake damping fell in one direction to 0.8% and increased in one direction to 3.6%. Measurements made during the Loma Prieta Earthquake showed that the damping increased to 11.6% and 15.5% in the translational directions. No information was given on whether the buildings showed physical damage.

Others researchers also studied the dynamic properties of two tall shear wall buildings in Hawaii (Taoka et al., 1974). The more relevant of the two was a 37-storey, 113 m tall RC coupled wall building. It had 5 pairs of 200 to 300 mm thick shear walls, two of which were coupled with lintel beams 450 mm wide and 510 mm deep. In the other direction there were three pairs of 250 mm walls, coupled with 450 mm wide and 510 mm deep coupling beams. The damping ratios were estimated to be 1% to 1.1% for the fundamental modes in the first direction and 1.1% to 1.2% in the second direction.

The University of Western Ontario has been conducting in-situ tests on a tall slender RC coupled wall building in downtown Toronto (Izymov et al., 2010). The building was expected to have major across-wind vibration problems due to its shape and has four Tuned Liquid Column Dampers (TLCD) installed at its top floors. The structure was completed recently and during the study the measurements had not yet captured a significant wind event and the building was also not fully occupied, therefore the concrete elements probably had not cracked and the full mass probably was not achieved. The measured short direction frequency and rotational frequency were calculated as 0.27 Hz and 0.54 Hz, respectively, while FE analyses predicted frequencies of 0.179 Hz and 0.318 Hz, respectively based on 10-year wind cracking estimates. There is a loss in effectiveness in the performance of TLCDs when tuned to frequencies different than actual, the authors suggest that the tuning can be readjusted over the life of the building. The
TLCD designers have suggested that the building be monitored for an additional 5 years to possibly re-tune the system in the future.

Recent measurements of the dynamic characteristics of the Taipei 101 building were published (Li et al. 2011). The Taipei 101 was the tallest building in the world when completed in 2003 and is equipped with a tuned pendulum damper. There were four large events that occurred during the monitoring period, including the Wenchuan earthquake and 3 typhoons. The measurements indicated that the damping of the structure was scattered and decreased for the higher modes. The damping in the fundamental frequency showed some amplitude dependent response characteristics and the damping in the two orthogonal directions varied from 0.77% to 3.74% for the different events. Also the fundamental period of the structure increased when the amplitude of vibration increased.
Amplitude-dependent characteristics were observed in a recent study from Fang et al. (1999) on a 120m tall composite building in Hong Kong. The low-amplitude plateau occurred at a value of $\xi_0 = 0.25\%$ for an amplitude of vibration at the top storey of 6.9mm with a high-amplitude plateau of $\xi_{max} = 0.49\%$ for an amplitude of vibration at the top storey of 15mm.

In another recent study, Li et al. (2002 and 2003) characterized the amplitude-dependent damping of the 79-storey Di-Wang Tower in Shenzhen, China, which has a lateral load resisting system consisting of an RC core and perimeter steel frames coupled with outrigger trusses at four levels. The low-amplitude plateau occurred at $\xi_0 = 0.1\%$ and the high-amplitude plateau occurred at $\xi_{max} = 0.46\%$, which is shown in Figure 2.9d. The authors concluded that damping levels assumed in current design and wind-tunnel studies were high and unconservative.

Areemit et al. (2010) conducted measurements on a 50-storey coupled wall building in downtown Toronto. There were two sets of three storey outriggers in both principal directions. Ambient vibrations were recorded during construction of the structure just prior to opening of the building. The damping properties exhibited significant scatter and the average was about $\xi_1 = 0.7\%$. The natural frequency of vibration was $f_1 = 0.41Hz$, about twice as large as the SLS FE model that was used for the design of the structure.

Three studies of high-rise RC buildings reported values of damping for the fundamental mode of vibration of: $\xi_1 = 0.60\%$ for the 391m CITIC Plaza in Guanzhou (Li et al. 2010), $\xi_1 = 1.40\%$ for the 319m Nina Tower in Hong Kong and $\xi_1 = 0.40\%$ for the 308m One Island East Tower in Hong Kong (Au et al. 2009).

Smith et al. (2010) conducted a survey on the damping values that structural engineers assumed at the design stage with respect to height. The equation that he proposed from the survey of structural engineers was, $\xi_1 = 1.7H^{-0.469}$, which showed that the damping assumed was larger than damping measured, especially for the taller buildings. Kareem and Gurley (1996), also reported design damping assumptions have coefficients of variation up to 70% compared to empirical predictions, generating significant uncertainties in the assessment of wind-induced response of these buildings.
Smith et al. (2010) offered possible reasons for damping decreasing with height from the recent measurements:

1) There is a base level of intrinsic material damping which is approximately 0.3%.

2) The size of the nonstructural components such as cladding and partitions does not increase for taller buildings while the size of the structural members does increase significantly. The relative contribution to the overall damping from these nonstructural components is predicted to be more for shorter buildings.

3) Foundations contribute to the structural damping more in shorter buildings than in taller buildings.

New Performance-Based Seismic Design guidelines (SEAONC 2007, CBTUH 2008 LATBSDC 2008 and PEER 2009) have suggested assuming damping values based on the observed recent lower inherent measurements and any additional damping from structural member yielding is included in nonlinear analyses.

2.2.3.1 Summary of Inherent Building Property Measurements

A few important conclusions on the damping and natural frequency values that have been measured on tall buildings are:

1) There is a distinct downward trend in damping as building height and period increase. Above 250m nearly all RC structures have measured values of damping below 1%.

2) There is significant scatter in the data that has been measured.

3) Higher modes of vibration have about 40% more inherent damping than the fundamental modes of vibration.

4) The assumed damping values used in current practice of 1.5-2% and 2-2.5% for the SLS and ULS loading conditions, respectively, are likely unconservative. This could result in significant underestimates of building wind-induced response. Wind-tunnel results are not reliable when assumed damping values are not realistic.

5) The natural frequencies of vibrations measured are larger than design levels, however many important factors were left out of the studies including if assumed member design cracking parameters had been achieved.

6) Over the service life of the building, the building is expected to be subject to many large load amplitudes. The gross effective stiffness properties of the concrete members decrease with an increase of the amplitude of loading.
CHAPTER 2: Behaviour and Design of RC Coupled Wall High-Rise Buildings

2.3 HIGH-RISE BUILDINGS SUBJECT TO WIND

2.3.1 Introduction

The behaviour of high-rise buildings subjected to wind loading is very dynamic in nature. Quasi-static wind loads give a false impression of the real behavior of high-rise buildings in wind (Davenport, 1967). High-rise buildings are constantly being buffeted by gusts and although they tend to deflect to a mean position in the along-wind direction, the building constantly vibrates and sways with amplitudes which may be larger than the mean deflection. Rathbun (1940) described observations of the motion of the Empire State building: “the building tended to vibrate continuously like the tines of tuning a fork”, illustrating that the building vibrated continuously and in resonance for the duration of a wind storm. Traces from the motion showed that the building oscillated primarily in the natural modes of vibration with little influence from higher modes. In general, as buildings become taller these dynamic oscillations become more and more significant, causing motion perception problems for the occupants and contributing greatly to the overall deflection and structural member forces.

Theoretical developments in predicting the response of high-rise buildings subject to wind are important, however more useful to designers is the understanding of the dynamic conditions of the wind, and how the forces and the dynamic stiffness and damping characteristics relate to the structures response. Also, wind-tunnel studies are a necessity for the design of these tall structures, since the flow of the wind in urban environments is quite varied due to upwind objects causing turbulence and the change of the wind characteristics over the height of the structure.

This section gives some insights into the method of analysis used by wind engineers which is based on random vibration theory. After that, the nature of wind loading is described and it’s dynamic characteristics are summarized. This is followed by a section on wind-tunnel testing methods. Finally building response will be studied, with a focus on how stiffness and damping properties influence the response. Stiffness and damping properties are focused on, because these are design properties which a structural engineer can have some control over. A more thorough discussion of the wind effects on buildings can be found in Simiu and Scanlan (1996) or Holmes (2007).

2.3.2 Random Vibration Approach

Davenport developed a procedure using random vibration theory to treat wind-induced vibrations of civil engineering structures, which has now become the standard in this industry and has been incorporated into many building codes (Davenport 1961, 1963 and Isyumov 1999). This technique is referred to as the random vibration or spectral approach.
Wind velocities, pressures and forces are modelled as random processes. This approach makes use of statistical quantities such as standard deviations, correlations and spectral densities to describe the wind forces and the structural response. Wind storms can never be predicted deterministically, but they can be defined well statistically.

The components of the wind-induced structural actions are divided into three main categories: i) the time-averaged mean or static loads, ii) the slowly varying fluctuating wind forces which are not amplified by the structure, commonly referred to as background loads, and iii) the resonant dynamic loads, where the action of the wind-induced forces is amplified by the structure due to resonance in one or several of its modes of vibration.

Figure 2.10 illustrates the steps utilized in the random vibration approach to determine the wind-induced dynamic response. The analysis is transformed from the time-domain into the frequency-domain utilizing the gust power spectral density, which is a commonly-used tool in signal processing. The velocity time-history (Figure 2.10a) is transformed into the wind gust power spectral density, which represents the statistical characteristics of the dynamic components of the wind (Figure 2.10d). The frequency dependent aerodynamic admittance function (Figure 2.10e) provides a link between the wind gust power spectral density and the aerodynamic force spectral density. The aerodynamic admittance function describes the influence of the size of the disturbance in relation to the size of the structure. The aerodynamic force spectral density (Figure 2.10f) describes the statistical features of the force time-history (Figure 2.10b). Lastly, the aerodynamic force spectral density is linked to the response spectral density (Figure 2.10h) by the mechanical admittance function (Figure 2.10g). The mechanical admittance function describes the influence of the dynamic properties of the building itself on the response characteristics. The response spectral density describes the response time-history of the structure (Figure 2.10c) statistically. The reader is directed towards Bendat and Peirson (2000) for random analysis theory and techniques.

2.3.3 Wind Characteristics

2.3.3.1 Variation of Wind Mean Velocity with Height

The mean wind velocity profile varies with height. At the ground surface, there is a retarding effect due to surface friction, causing the wind speed to be negligibly small. The wind velocity increases curvilinearly from zero at the ground level to a maximum (gradient velocity) at the gradient height. The increase in wind velocity is affected by the upstream terrain over which it is blowing, with the gradient height being significantly lower in rural areas compared to urban centers. At a height of about 300 m in open country and 500 m above ground in urban terrain the wind speed is virtually unaffected from surface friction and is only dependent on prevailing seasonal and local wind effects. This height is known as the gradient height $z_g$. The height between the ground surface and the gradient height, where the wind speeds...
are affected by topography, is known as the atmospheric boundary layer (ABL). The mean wind speed at a height \( z \), \( \bar{V}(z) \), can be predicted using a simple power-law representation:

\[
\bar{V}(z) = \bar{V}_g \left( \frac{z}{z_g} \right)^\alpha
\]

(2.14)

where \( \bar{V}_g \) is the gradient wind speed assumed constant above the boundary layer and \( \alpha \) is a power law coefficient varying from about 0.16 in open country to 0.4 in urban areas. This is illustrated in Figures 2.11a and 2.11b.

Figure 2.10 Random Vibration Approach to Resonant Dynamic Wind Response (adapted from Davenport, 1963)

Figure 2.11 Wind Velocity Profile with Height

(a) Open Profile  (b) Urban Profile  (c) Velocity History with Height (after Deacon, 1955)
2.3.3.2 Wind Turbulence

Superimposed on the mean wind speed is turbulence, which is caused by friction when wind flows over obstacles. Turbulence is always present in the atmospheric boundary layer. Figure 2.11c displays wind speed records at three locations of different heights on a tall mast in open terrain during a typical strong wind storm.

The records show that the total wind speed at a given time can be approximated as a summation of a mean component and a fluctuating component (Davenport 1987). The mean wind speed increases with height, but is approximately constant over the duration of the wind storm. The gust amplitude is approximately constant with height. Obstacles introduce random vertical and horizontal movements causing changes both in speed and direction. The velocity of wind is expressed in vector form as:

\[ V(z, t) = V(z) + v'(z, t) \hat{i} + w'(z, t) \hat{j} + r'(z, t) \hat{k} \]

where \( v' \), \( w' \) and \( r' \) are the fluctuating components of the gust in the \( x \), \( y \) and \( z \) directions, respectively.

2.3.3.3 Spectrum of Turbulence

Davenport (1961) used statistical tools to represent wind behavior. He applied the power spectrum to wind velocity which represents the power or kinetic energy per unit time, associated with fluctuations in the wind at different frequencies. Davenport looked at very long-term wind speed records in the frequency domain and noticed the energy content can be separated into two distinct components, gust events and mean wind events that are separated by a distinctive frequency gap, which is graphically illustrated in Figure 2.12a.

Wind is rich in energy for a wide range of frequencies with a spectral gap centered at about 10-60 minutes. To the right of the spectral gap is the high frequency region which is associated with gust turbulence and to the left is the low frequency region which is associated with mean wind. The high frequency region has a peak at approximately 1 cycle per minute, the mean wind region has a peak at approximately 1 cycle every 4 days and a smaller peak at one cycle every day. The high frequency peak is shifted to the right in urban areas because of the increased number of obstacles that cause increased turbulence. Also, the gust area is larger for strong winds compared to light winds. An average period of 10 minutes to an hour typically provides stable mean wind speed values for wind storm records.
The spectral gap shows that the wind climate and turbulence in the atmospheric boundary layer are mutually independent, so they can be treated separately and superimposed. Storm turbulence has constant statistical features from periods above 10 minutes to an hour. The wind velocity statistical properties at any time in the ensemble are not dependent on time separation between measuring points. This is the case for a typical storm, where the mean, variance, root mean square and standard deviation of wind velocity do not vary with time. Also, any time portion wind-time history average must be equal to the total ensemble average. The variance of the signal, $\sigma_v^2$, can be calculated from the turbulent time-history as:

$$\sigma_v^2 = \frac{1}{T} \int_0^T (v'(t))^2 dt$$  \hspace{1cm} (2.16)

Analogous to Eq. (2.16), the wind time-history can be transformed into the frequency domain and expressed as the wind gust spectral density, $S_v(f)$, at a frequency, $f$. This can be used to determine the variance of the wind velocity ensemble, $\sigma_v^2$, which is the area underneath the $S_v(f)$ versus $f$ graph in Figure 2.10d:

$$\sigma_v^2 = \int_0^\infty S_v(f) df$$  \hspace{1cm} (2.17)

**Figure 2.12 Wind Power Spectrums**
Davenport developed a mathematical expression for the turbulent gust wind spectrum, which is the basis for the National Building Code of Canada, NBCC 2005 (NRC 2006), wind loading expressions. The normalized spectrum, \( \frac{fS_v(f)}{\sigma_v^2} \), can be expressed as:

\[
\frac{fS_v(f)}{\sigma_v^2} = \frac{2}{3} \left( \frac{fL}{V_{10}} \right)^2 \left( 1 + \left( \frac{fL}{V_{10}} \right)^2 \right)^{-\frac{3}{4}}
\]

(2.18)

where \( L = 1200\,m \) and \( V_{10} \) is the mean velocity at a height of 10\,\text{m} . Figure 2.12b displays the normalized Davenport spectrum for the 1 in 50 year wind speed in Toronto at a reference height of 10\,m.

High-rise buildings have different velocity characteristics up the height of the structure. Typically wind engineers assume high-rise buildings are line-like structures and using a coherence function they can determine the cross-correlation of signals up the height of the structure (Davenport 1963).

2.3.3.4 Comparison of Statistical Properties of Wind and Earthquakes

There are distinct differences between dynamic excitation of structures caused by wind and those caused by earthquakes, which is outlined in more detail in Holmes (2007). The Normalized Power Spectral density of an earthquake and a wind storm are illustrated in Figure 2.13a. The duration of an earthquake is generally much shorter than a wind storm, approximately 1 min, compared to 1 hour for a typical wind storm (Simui and Scanlan 1996). Earthquakes are treated as transient loadings in the time domain where as dynamic wind is generally treated as random vibrations in the frequency domain. The predominant frequencies of earthquake vibrations are generally 10-50 times larger than the predominant frequencies in fully developed wind-storms and therefore some buildings may be more prone to earthquake excitations than wind excitations and vice versa. Earthquake ground motions cause fully-correlated equivalent forces over the height of the structure whereas wind loads are only partially correlated over the height of the structure due to the turbulence generated in strong wind storms.

2.3.3.5 Bluff Body Forces

Bluff body forces are caused by wind pressures developed when the wind strikes the face of a high-rise building. This section illustrates how the mean time-averaged force and the dynamic force are developed in typical wind codes.
a) Power Spectral Density Wind vs EQ (Holmes, 2010)  

b) Arbitrary Forces in 2-D Flow

Energy Content  
(Normalized Power  
Spectral Density)

Figure 2.13 Normalized Power Spectral Density and Arbitrary Bluff Body Forces

There have been large theoretical advancements in the field of bluff body aerodynamics, however the majority of concepts are still empirical and have been developed from experimental studies. Figure 2.13b displays an arbitrary bluff body subject to steady free-stream 2-dimensional flow. The lift force, drag force and torque per unit height of a rectangle are expressed as:

\[
F_D = \frac{1}{2} \rho C_D D \bar{V}^2
\]  

\[
F_L = \frac{1}{2} \rho C_L D \bar{V}^2
\]  

\[
F_T = \frac{1}{2} \rho C_T D^2 \bar{V}^2
\]

where \( D \) is the depth, \( \rho \) is the density of air and \( \bar{V} \) is the mean wind. \( C_D, C_L \) and \( C_T \) are the drag, lift and torque coefficients. The pressure coefficients in turbulent flow for high-rise buildings are functions of the shape of the building, turbulence, angle of attack, Reynolds number and distribution of wind velocity over the height of the building. The velocity in the along-wind direction can be rewritten from Eq. (2.15) as:

\[
V(t) = \bar{V} + v'(t)
\]
The time-dependent along-wind force can be expressed as Eq. (2.19):

\[
F_D(t) = \frac{1}{2} \rho C_D A [\overline{V} + v'(t)]^2 = \frac{1}{2} \rho C_D A [\overline{V}^2 + \overline{V} v(t) + v^2(t)]
\]  

(2.23)

where \( A \) is the bluff body area, \( v^2(t) \) is small relative to the other terms in Eq. (2.23) and is therefore typically neglected. The drag force is expressed as a combination of mean, \( F_D \), and fluctuating components, \( F_D'(t) \) as:

\[
F_D(t) = F_D + F_D'(t)
\]

(2.24)

where:

\[
F_D = \frac{1}{2} \rho C_D A \overline{V}^2
\]

(2.25)

\[
F_D'(t) = \rho C_D A \overline{V} v(t)
\]

(2.26)

and \( F_D \) is the mean component of the force and \( F_D'(t) \) is the time-varying dynamic component.

In urban environments, wind is extremely turbulent in nature and the previous equations cannot predict all of the expected scenarios. Figure 2.14a displays massively separated flow on the leeward side of a tall building. The relatively laminar flow on the windward side becomes extremely turbulent after it strikes the front face of the building. Figure 2.14b displays a different tall building in a typical boundary layer flow profile and the mean time-averaged pressure distribution over the height. As can be seen at the back face of the building the pressure is almost evenly distributed, but at the front face of the building the pressure profile increases with height.

From these discussions it can be seen that even for simple, regular shapes in typical boundary layer flow profiles the mean pressure distribution is difficult to predict and turbulent flow is extremely irregular. This still doesn’t include the dynamic wind forces or the dynamic response of the building caused by the building vibrating back and forth in resonance. For this reason, many buildings are modelled in the wind tunnel using scaled studies with the actual geometry of the building and including its immediate surroundings. Since this is the case, current design is based on results obtained from the wind tunnel. The remainder of this section will discuss wind tunnel studies and how this relates to the response of high-rise buildings.
2.3.4 Wind Tunnel Model Testing

2.3.4.1 Introduction

The High-Frequency Force Balance (HFFB) technique (Tschanz and Davenport 1983) represented a breakthrough in wind engineering and is now the most common method for testing tall buildings. Prior to the technique being developed, aeroelastic models were the most common type of building wind tunnel tests. Aeroelastic models are fully dynamically scaled structures, which have the representative scaled geometric, stiffness, mass and damping properties of the actual structure. Aeroelastic models can measure the influence of the structures’ dynamic properties on the incoming wind flow and the interplay between the two, which is commonly referred to as aeroelastic feedback. Aeroelastic feedback is very important for vortex shedding of the along-wind loads, across-wind accelerations and torsional velocities in tall buildings. Unfortunately, these models take a very long time to build and if there are significant changes to the building structural configuration the tests have to be conducted again. Now aeroelastic models are typically only used for unique structures that operate in “uncharted waters” such as the current world’s tallest tower, the Burj Khalifa.
2.3.5 High-frequency Load Balance Test

The high-frequency load balance test consists of a reduced scale model which is a lightweight, stiff, geometrical representation of the building attached to an ultra-sensitive force balance. The force balance is usually made of high-strength steel rods with strain gauges mounted near the base. The base moments are measured directly from strain gauges mounted on the steel rods. The structure and surroundings are typically 1 to 400 scale. The model building model frequency is made much larger than the excitation frequency so that there is no resonant amplification of the dynamic force. The forces measured on the base balance are the basis for the wind force spectral density, $S_c(f)$, and the variance of the force, $\sigma^2_c$. This removes the uncertainty associated with the shape of the building and the turbulent upstream wind flow. This technique approximates the fundamental sway and torsional steady and unsteady modal forces using model analysis techniques. Amplification factors are derived analytically (mechanical admittance functions) for the natural modes of vibration using random vibration analysis methods and then used to estimate the entire building displacement response wind spectral density, $S_u(f)$, and the variance of the displacement $\sigma^2_u$. For a given exterior geometry of a building, multiple results can be obtained analytically for different structural configurations and properties from the first experimental results.

There are two separate test procedures. The first measures mean and rms base bending moments along the orthogonal building axes in the fundamental sway and torsional modes. The second records the spectra of base loads, which are used to determine the turbulence-induced unsteady forces at the natural frequencies of vibration to determine the resonant component of the response. Both of the procedures take measurements at 10° intervals for the full 360° azimuth range.

During the wind excitation of tall buildings, the main components of response come from each of the fundamental modes of vibration, excited by their respective modal forces. There is a mechanical transfer function which relates the forces exerted on the structure to the structures’ response. Without wind tunnel tests, the aerodynamic transfer function, relating the gust properties to the wind induced forces is difficult to establish.

Figure 2.15 illustrates some examples of wind-tunnel studies. Figure 2.15a shows a picture of a condominium development in downtown Toronto and Figure 2.15b shows wind tunnel models of the development and the surrounding built-up environment plus future planned projects. The studied buildings are red and future planned developments are green. Figure 2.15c displays a typical HFFB used for wind tunnel testing. It consists of an extremely stiff and lightweight model of the actual building with a highly sensitive stiff force balance which measures the overturning moments.
CHAPTER 2: Behaviour and Design of RC Coupled Wall High-Rise Buildings

2.3.6 High-Frequency Force Balance (HFFB) Procedure

The measurements obtained during the force balance tests are the shear forces, \( V_x(t) \), \( V_y(t) \), the base moments, \( M_x(t) \), \( M_y(t) \) and the base torque, \( T(t) \), at time, \( t \). The generalized modal forces are related to the base moments through approximate linear mode shapes, \( \phi_i \), relative to the center of coordinates, \( \phi_i(z) = a_i(z/H) \), for each sway mode, \( i \). The generalized force, \( F_i(t) \), for each mode, \( i \), is obtained by (BLWTL 1999):

\[
F_i(t) = (a_{xi}/H)M_x(t) + (a_{yi}/H)M_y(t) + 0.7a_{0i}T(t)
\]  \hspace{1cm} (2.27)

where \( H \) is the height of the building and \( a_{xi}, a_{yi} \) and \( a_{0i} \) are the “modal mixing factors” in each of the horizontal directions and the torque response. The generalized mass, \( M_i \), is calculated for each sway mode:

\[
M_i = \sum_j (m_j\phi_{xij}^2 + m_j\phi_{yij}^2 + I_j\phi_{\theta ij}^2)
\]  \hspace{1cm} (2.28)

where \( m_j \) and \( I_j \) are the mass and mass moment of inertia at floor \( j \). The generalized stiffness, \( K_i \), is calculated for each sway mode:

\[
K_i = (2\pi f_i)^2 M_i
\]  \hspace{1cm} (2.29)
The actual building behavior is composed of both the mean static response and the fluctuating
dynamic response. The mean static deflection, \( \bar{u}_j \), can be obtained directly from the test for any storey
location \( z_j \):

\[
\bar{u}_{ij} = \sum_i \left( \frac{F_i}{K_i} \right) \phi_i(z_j)
\]  
\hspace{1cm} (2.30)

\[
\bar{u}_{yj} = \sum_i \left( \frac{F_i}{K_i} \right) \phi_{yi}(z_j)
\]  
\hspace{1cm} (2.31)

\[
\bar{u}_{\theta j} = \sum_i \left( \frac{F_i}{K_i} \right) \phi_\theta(z_j)
\]  
\hspace{1cm} (2.32)

The time-varying component response is a combination of a slow varying, quasi-static or
background response and a dynamic or resonant response in the first few modes of vibration. The dynamic
response is calculated using the mechanical admittance function at the natural frequencies of vibration.
Using the mechanical admittance function with the force power spectrum yields the response power
spectrum that is illustrated in Figure 2.16a. In the figure, the sharp spikes around the natural frequencies of
the system, identify the narrow-band or resonant response. The broad hump is governed by the shape of
the force spectral density, termed the slow varying broad-band or background response. The total variance
of the displacement response is calculated as:

\[
\sigma_u^2 = \sigma_B^2 + \sigma_R^2
\]  
\hspace{1cm} (2.33)

The background response is related to the quasi-static response that is caused by gusts below the natural
frequencies of the structure and is independent of the natural frequency of the structure. The calculation of
the resonant response assumes that over the resonant peak in Figure 2.16a the function \( S_r(f) \) is constant at
\( S_r(f_1) \) and \( S_r(f_2) \), which is valid for the flat spectral densities for wind loading and when the resonant peak
is narrow, as with high-rise structures with low damping (Ashraf Ali and Gould 1985). For many structures,
the background response is dominant, however for high-rise buildings with long natural periods of
vibration and low damping values the resonant response can be dominant. Figures 2.16c and 2.16d
illustrate the difference in the response of a structure with a high natural frequency, such as a short building
(Figure 2.16c) and a structure with a low frequency, such as a high-rise building (Figure 2.16d) subject to
the drag force time-history shown in Figure 2.16b. Both Figures 2.16c and 2.16d follow the slow varying
changes in the drag force in Figure 2.16b, but in Figure 2.16d the large amplification of the resonant
response of the low frequency structure can be clearly seen.
The variance of the displacement response, \( \sigma_{u_j}^2 \), for any level \( z_j \) can be expressed as:

\[
\sigma_{u_j}^2 = \sum_{i} \left[ \frac{\sigma_{F_i}}{K_i} \phi_{i}(z_j) \left( 1 + \frac{\pi f_i S_{F_i}}{4 \xi_i \sigma_{F_i}} \right) \right]^2
\]  
(2.34)

\[
\sigma_{u_j}^2 = \sum_{i} \left[ \frac{\sigma_{F_i}}{K_i} \phi_{j}(z_j) \left( 1 + \frac{\pi f_i S_{F_i}}{4 \xi_i \sigma_{F_i}} \right) \right]^2
\]  
(2.35)

\[
\sigma_{u_j}^2 = \sum_{i} \left[ \frac{\sigma_{F_i}}{K_i} \phi_{i}(z_j) \left( 1 + \frac{\pi f_i S_{F_i}}{4 \xi_i \sigma_{F_i}} \right) \right]^2
\]  
(2.36)

where \( \sigma_{F_i} \) is the root mean square (rms) generalized force, \( f_i \) is the frequency, \( S_{F_i} \) is the response spectrum and \( \xi_i \) is the damping for mode \( i \). The first term in Eqs. (2.34), (2.35) and (2.36) represents the background component and the second term represents the resonant response.
The acceleration responses are calculated using only the resonant vibration component. The variance of the acceleration is expressed as:

\[ \sigma^2_{u_{ij}} = \sum_i \left( 2\pi f_i \right)^2 \frac{\sigma_{F_i}^2 \phi_{x_i}(z_j)}{K_i \sqrt{i \frac{\pi f_i S_{F_i}}{4 \xi_i^2 \sigma_{F_i}^2}}}^2 \]  

(2.37)

\[ \sigma^2_{u_{ij}} = \sum_i \left( 2\pi f_i \right)^2 \frac{\sigma_{F_i}^2 \phi_{y_i}(z_j)}{K_i \sqrt{i \frac{\pi f_i S_{F_i}}{4 \xi_i^2 \sigma_{F_i}^2}}}^2 \]  

(2.38)

\[ \sigma^2_{u_{ij}} = \sum_i \left( 2\pi f_i \right)^2 \frac{\sigma_{F_i}^2 \phi_{\theta_i}(z_j)}{K_i \sqrt{i \frac{\pi f_i S_{F_i}}{4 \xi_i^2 \sigma_{F_i}^2}}}^2 \]  

(2.39)

The variance of the torsional velocity \( \sigma^2_{u_{0j}} \) can be calculated:

\[ \sigma^2_{u_{0j}} = \sum_i \left( 2\pi f_i \right)^2 \frac{\sigma_{F_i}^2 \phi_{0_i}(z_j)}{K_i \sqrt{i \frac{\pi f_i S_{F_i}}{4 \xi_i^2 \sigma_{F_i}^2}}}^2 \]  

(2.40)

The variance of the resonant base moments are calculated using inertia loads that are located at the center of masses, therefore eccentricities \( e_x \), \( e_y \) and \( e_{\theta} = e_x^2 + e_y^2 \) need to be included in base moment calculations. The variance of the resonant base moments are calculated below:

\[ \sigma^2_{M_x(Resonant)} = \sum_i \left\{ \frac{\left[ \frac{\pi f_i S_{F_i}}{4 \xi_i^2 \sigma_{F_i}^2} \sum_j \{ (m_j \phi_{x_{ij}} - e_{x_j} m_j \phi_{0_{ij}}) z_j \} \right]^2}{M_i} \right\} \]  

(2.41)

\[ \sigma^2_{M_y(Resonant)} = \sum_i \left\{ \frac{\left[ \frac{\pi f_i S_{F_i}}{4 \xi_i^2 \sigma_{F_i}^2} \sum_j \{ (m_j \phi_{y_{ij}} - e_{y_j} m_j \phi_{0_{ij}}) z_j \} \right]^2}{M_i} \right\} \]  

(2.42)

\[ \sigma^2_{T(Resonant)} = \sum_i \left\{ \frac{\left[ \frac{\pi f_i S_{F_i}}{4 \xi_i^2 \sigma_{F_i}^2} \sum_j \{ -e_{\theta_j} m_j \phi_{x_{ij}} + e_{x_j} m_j \phi_{\theta_{ij}} + (I_{ij} + e_{\theta_j} m_j) \phi_{0_{ij}} \} \right]^2}{M_{ij}} \right\} \]  

(2.43)
The background rms and mean base moments are obtained directly from the test. The peak loads and responses are determined from:

\[ R = \overline{R} + g_f \sigma_R \]  \hspace{1cm} (2.44)

where \( R \) represents any loads or responses including base moments, deflections, accelerations, torsional velocity or other loads, \( \overline{R} \) is the mean value and \( \sigma_R \) is the dynamic root mean squared component and \( g_f \) is the gust factor. The Boundary Layer Wind Tunnel Laboratory (BLWTL 1999) and typically uses values of 3.7 to 3.8 for the gust factor, \( g_f \). Resultant centroidal and corner accelerations are computed to determine the maximum response.

Effective static wind loads for each storey are then determined and applied up the height of the structure. Typically twenty-four load combinations are generated in order to be used with the effective storey wind loads and to be used in a FE model. The 24 load combinations are a function of meteorological probability and peak loads on a direction-by-direction basis from the wind tunnel study and represent an envelope of all possible wind conditions.

The current practice involves multiple exchanges and iterations between the structural engineers and wind engineers. The maximum lateral accelerations and torsional velocities are determined using the analytical dynamic techniques and the wind tunnel base moment results. The human perception criteria is then evaluated.

The wind engineers provide the structural engineers with the new wind loads and wind combinations with the structural FE model to obtain the maximum drift envelopes and member force envelopes. These are then checked against the strength and drift criteria design criteria.

Often the human perception, drift or strength criteria are not met and the structural engineer has to convince the architect to change the structural layout or increase the structural member sizes. This iterative process takes a significant amount of time. For every new structural configuration the FE model is modified producing new modal properties. These new properties are then given to the wind engineer which are then used to compute a new set of wind tunnel results. If the design limits are not easily achievable then the only other option is to reduce the number of stories or add a vibration absorber. For these reasons the structural high-rise designer is heavily reliant on wind tunnel results.
2.3.7 Dependency of High-Rise Building Response on Dynamic Properties

The response of high-rise buildings to wind depend on the damping, stiffness and to a lesser extent the mass of the buildings. The response is also affected by the shape of the structure, however, structural designers generally do not have much input on the mass or shape. Vickery et al. (1983) approximated the dynamic response of structures to wind loads as functions of damping, stiffness and mass. The approximate expressions that were suggested are given in Table 2-2:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Expression</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resonant Along-Wind Load</td>
<td>$F_{\text{Along (Resonant)}} \approx c_1(M/(K\xi))$</td>
</tr>
<tr>
<td>Along-Wind Accelerations</td>
<td>$\ddot{u}_{\text{Along}} \approx c_2/(\sqrt{MK\xi})$</td>
</tr>
<tr>
<td>Resonant Across-Wind Load</td>
<td>$F_{\text{Across (Resonant)}} \approx c_3(M/(K\sqrt{\xi}))$</td>
</tr>
<tr>
<td>Across-Wind Accelerations</td>
<td>$\ddot{u}_{\text{Across}} \approx c_4/\sqrt{\xi}$</td>
</tr>
</tbody>
</table>

where $c_1, c_2, c_3$ and $c_4$ are constants. As can be seen from the expressions, damping has significant implications on the response of structures to dynamic loads.

To demonstrate some of the benefits of added damping versus stiffness in the dynamic response of high-rise buildings, two case studies are illustrated. The results are obtained from the dynamic procedure in the NBCC 2005 (NRC 2006), which provides closed-form approximations of the wind response of regular structures.

The first case study is a symmetric, square 40-storey concrete building in downtown Toronto. The period is varied from $T = 3s$ to $5s$ for six damping values, $\xi_1 = 0.5, 1, 1.5, 2, 3$ and $5\%$. The second is a symmetric, square 85-storey concrete building in downtown Toronto. The period is varied from $T = 5.5s$ to $8.5s$ for the same damping values. The case study building properties are given in Table 2-3 and the results according to the NBCC 2005 dynamic procedure for the 40-storey and 85-storey building, are located in Figures 2.17 and 2.18, respectively.

<table>
<thead>
<tr>
<th>Case Study</th>
<th>Height (m)</th>
<th>Width (m)</th>
<th>Depth (m)</th>
<th>$m_{\text{Storey}}$ (tonne)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40-Storey</td>
<td>120</td>
<td>30</td>
<td>30</td>
<td>840</td>
</tr>
<tr>
<td>85-Storey</td>
<td>265</td>
<td>40</td>
<td>40</td>
<td>1500</td>
</tr>
</tbody>
</table>
As can be seen in Figures 2.17 and 2.18, increasing the damping reduces the base shear, top storey accelerations and deflections, whereas increasing the period increases the response of all parameters. Looking at the displacements it can be seen that both the damping and the period have large effects. The reason the along-wind displacement increases significantly when the period increases is because of the loss of stiffness in both the static and dynamic components of the response. The along-wind base shear is the least affected by the damping and stiffness, because it is response quantity mostly based on the static or non-dynamic response. The 85-storey structure base shear is more dependent on the dynamic building properties, which is due to the resonant amplification of wind loads that are more prevalent in taller structures with longer periods. As can be seen in the figures the acceleration in both the along-wind and across-wind directions are significantly affected by the damping and also the period. This is due to the fact that the acceleration response is entirely dynamic. For both buildings, an increase in damping and a decrease in period lead to a reduction of accelerations. For such low levels of damping in high-rise
buildings, small increases in damping result in large reductions in accelerations. The along-wind and across-wind accelerations are relatively the same for the 40-storey structure for the range of damping and stiffness that was considered, but in the 85-storey structure the acceleration in the across-wind direction is much more. Incorrect assumptions in the damping at the low levels of damping expected in tall buildings can lead to structures being significantly more wind sensitive than assumed at the design stage, potentially leading to i) lower factors of safety, ii) fatigue issues and iii) serviceability issues such as perceived occupant accelerations.

Figure 2.18 85-Storey Building
2.4 WIND DESIGN PHILOSOPHY

One of the primary concerns for designers of high-rise buildings is the choice of the lateral load resisting system and the resulting performance of the structure under wind induced loads and vibrations. In Canada, the 1 in 10 year wind loads are used for checking the serviceability deflection and occupant comfort criteria (SLS), and the 1 in 50 year wind loads are used for the member ultimate limit strength criteria (ULS) (NRC 2006). Since large wind storms are expected to occur multiple times during the life of a structure, high-rise buildings are designed to remain essentially linear elastic for the 1 in 50 year (ULS) loading conditions.

\[ \phi_n U_n \geq \alpha_i L_i \]  

(2.45)

where \( U_n \) is the nominal capacity of members, \( \phi_n \) is the strength reduction factor to account for uncertainty in the capacity for all structural members and failure modes, \( n \), \( \alpha_i \) is the load factor and \( L_i \) is the load demand for all loads, \( i \), that are considered in the various design load combinations. This criteria must be met for all structural members.

The usual design approach is to increase the structural member sizes until the deflection and serviceability limits are met. This is essentially a stiffness based design procedure. On the other hand, architects and owners want to minimize the sizes of structural members to maximize usable architectural space. In many cases, solely increasing the sizes of structural members does not lead to a design that meets the SLS criteria without a complete redefinition of the lateral load resisting system. Such a change in the design of the structure can jeopardize the economic feasibility of a project. When this is the case, generally a vibration absorber system is incorporated into the top stories of the structure, as will be discussed in Section 2.6. However, the space occupied by such systems represents the most profitable locations for the developer and therefore result in a significant overall cost.

Clause 4.1.8.2 of the NBCC 2005 (NRC 2006) states that “any building whose height is greater than 4 times its minimum effective width or greater than 120 m tall and other buildings whose light weight, low frequency and low damping properties make them susceptible to vibrations shall be designed by experimental methods for the danger of overloading, vibration and the effects of fatigue, or by the dynamic approach to account for the action of wind gusts”. Experimental methods refer to wind tunnel testing and the dynamic approach refers to the dynamic code based analysis methods for typical structures that is included in Commentary I. Prudent designers engage wind tunnel engineers to conduct wind tunnel testing for high-rise buildings over 30 stories. Wind tunnel tests give a better understanding of the loading on the buildings based on more accurate information on the building surroundings and shape and on the across-wind and torsional wind response which are critical to occupant comfort.
2.4.1 Service Limit State Design Strategy

2.4.1.1 Drift Limits

The NBCC 2005 (NRC 2006) states that the storey drift of a structure subject to specified gravity and wind loads shall not exceed 1/500 of the storey height (Clause 4.1.3.5(3)). The storey drift is minimized mainly to ensure that glass and exterior non-structural panels are not damaged, although higher drift levels are allowed if it has been established that drifts will not damage non-structural elements (Appendix A-4.1.3.5.(3)). Also, excessive storey drifts may damage masonry or interior finishes. Commentary I states that if precautions are not taken to permit movement of interior partitions without damage, a maximum lateral deflection limitation of 1/250 to 1/1,000 of the building height should be observed.

For a 1 in 10 year wind storm, the ASCE/SEI 7-05 (ASCE 2006) has an absolute drift limit of 10 mm, for any given storey, independent of the height, and the storey drift limit should not exceed 1/400 to 1/600 of the storey height unless special details allow non-structural partitions, cladding, or glazing to accommodate larger drifts (Commentary CC.1.2) (ASCE 2006). Some building codes, such as the International Building Code, have less stringent storey drift limits of 1/400, and they allow the engineer to use their judgement to justify their design if the limit is not met (ICC 2009).

It is interesting to note that interstorey drift limits were developed for shear deformation type buildings, where the columns of each storey undergo double curvature flexural deformations. Coupled wall buildings deform more like cantilevers under wind loads, therefore the traditional interstorey drift limits may not accurately capture the level of deformation in a storey. Miranda and Akkar (2006) propose alternative drift limit strategies for buildings without shear deformation type behavior. The CTBUH (2008) suggests that drifts should be separated into two components: i) the overall flexural deformations, with tension and compressive deformations in columns and walls, that cause no damage and ii) shear (racking) deformations that are angular in-plane rotations of wall or cladding elements, that cause damage.

2.4.1.2 Human Perception Criteria

Occupant discomfort is affected by the degree of stimulation of the body’s central nervous system and the visual perception of building oscillations, which is especially noticeable for torsional movements. There is large variability between different individual responses and therefore occupant perception criteria are subjective to some degree. Motion in high-rise buildings can induce a large range of responses ranging from anxiety to seasickness or nausea, which can result in an otherwise acceptable structure obtaining a reputation that can cause difficulties in renting floor space (Smith and Coull 1991). Feld (1968) described a 55-storey building in New York that experienced disturbing oscillations during strong storms, “enough to make it impossible to write at a desk located in the top few floors, so that employees are regularly excused during such storm periods”. The actual human response is dictated by many factors including the
perception of the motion of suspended objects, noise due to turbulent wind, objects at a distance moving slightly by twisting, motion due to rapid changes in acceleration (i.e. jerks), the type of activities the occupant is performing and the frequency of the motion (i.e. perception of motion increases at higher frequencies of vibration) (Chen and Robinson 1972, Chang 1973, Hansen et al. 1973, Kareem 1982, Goto 1983, Jeary et al. 1988, Burton et al. 2006. and Tamura et al. 2006).

Hansen et al. (1973) investigated occupant's reactions in two 40-storey office buildings (approximately 168 m in height) in different cities, each with about 2,800 occupants subject to two large winter storms. Each building's dynamic properties and the building motion response intensity and duration are given in Table 2-4. An approximate peak acceleration is obtained by multiplying the rms acceleration by a gust factor of 3.5. The peak acceleration of the second building would be close to the acceleration perception criteria for a commercial building. After the storms, the occupants of the top one-third of the building and owners were interviewed. The different motion cues described by the occupants are given in Table 2-5. They include the perceived motion of objects, creaking sounds, the perceived motion of other buildings (caused by torsion), motion sickness (including headaches, nausea, dizziness and queasiness), comments from co-workers or other (elevator noise, wind whistling). The number of people objecting to a storm of that magnitude occurring once a year were 2% for building 1 and 12% for building 2. Three prominent developers and one structural engineer were asked “what percentage of the people in the top one-third of the building can object to the sway motion each year and not seriously affect your renting program”. Two of the developers answered 2% and the engineer and one developer answered 2 to 5%.

| TABLE 2-4 Building Dynamic Properties and Motion Intensity and Duration of two 40-Storey Buildings (after Hansen et al. 1973) |
|---------------------------------------------------------------|----------|-----------|-----------|
| Parameter | Unit | Building 1 | Building 2 |
| Period of Vibration (NS) | s | 5.9 | 4.2 |
| Period of Vibration (EW) | s | 5.3 | 4.2 |
| Period of Vibration (Torsion) | s | 5.4 | 2.7 |
| Damping (NS) | % | 1.8 | 2.0 |
| Damping (EW) | % | 0.9 | 1.5 |
| Damping (Torsion) | % | 0.8 | NA |
| Duration of detectable motion during work day | hr | 6 | 5 |
| Duration of Storm Peak | min | 30 | 20 |
| Top Building Floor Average rms acceleration during perceptible period | milli – g | 1 | 2 |
| Approximate Peak Acceleration during perceptible period | milli – g | 3.5 | 7.5 |
| Top Building Floor Average rms acceleration during storm peak | milli – g | 2 | 5 |
| Approximate Peak Acceleration during storm peak | milli – g | 7.5 | 17.5 |
Chang (1973) made recommendations for comfort limits that are now commonly referenced based on data from vibration frequencies higher than 1 Hz and extrapolated them into lower frequencies that are expected for high-rise structures, which are shown in Table 2-6.

### TABLE 2-6 Criteria for Occupant Comfort (after Chang 1973)

<table>
<thead>
<tr>
<th>Comfort Limit</th>
<th>Acceleration ( milli - g )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Not Perceptible</td>
<td>&lt; 0.5</td>
</tr>
<tr>
<td>Perceptible</td>
<td>0.5-1.5</td>
</tr>
<tr>
<td>Annoying</td>
<td>1.5-5</td>
</tr>
<tr>
<td>Very Annoying</td>
<td>5-15</td>
</tr>
<tr>
<td>Intolerable</td>
<td>&gt; 15</td>
</tr>
</tbody>
</table>

A recent study by Tamura and others (Tamura et al. 2006) was used to form the basis of the human perception criteria for the Architectural Institute of Japan (AIJ) Guidelines for Evaluation of Habitability to Building Vibration (AIJ 2004). The tests were conducted with people in sitting posture, subjected to fore-and-aft, side-to-side, elliptical and circular motion for sinusoidal and random excitations. For the low frequency range of 0.125-0.315 Hz, 47 people were subjected to sinusoidal motions and 61 to random excitations to simulate wind vibrations for 100 minutes, in an enclosed room excited using a shake-table. The subjects indicated at what horizontal motion amplitudes they perceived motion, and expressing it statistically the authors developed the probabilistic scale for perception criteria, that is illustrated in Figure 2.19. The figure indicates the percentage of subjects perceiving the motion under various loading amplitudes and frequencies. One interesting result was that subjects did not sense much difference between the random motion and the sinusoidal motion nor the direction of applied motion.
Due to the fact that occupant perception of accelerations is uncertain, acceptable levels vary substantially between different codes and practitioners. The first building code to give guidance on perception was the NBCC, which suggested that for a 10 year return period, the acceleration levels should be less than $10 - 30 \text{ milli-} g$, with the upper limit applicable to office buildings and the lower limit applicable for residential buildings (Commentary I-34 (78)). Past North American wind tunnel studies have suggest limits of $10 - 15 \text{ milli-} g$ for residential buildings and $20 - 25 \text{ milli-} g$ for office buildings, based on a 10 year return period wind loading. Research has indicated that people’s sensitivity to motion is enhanced as the frequency of vibration is increased. The International Standards Organization (ISO 1997) has an rms acceleration limit for a 5 year return period:

$$a_{RMS_{max}} \approx 3.18g f^{0.412}$$

(2.46)

where $f$ is the building natural frequency in Hz and $g$ is a peak factor; Davenport (1964) recommended a peak factor of $g = \sqrt{2\log(vT)} + 0.577/\sqrt{2\log(vT)}$, where $v$ is the ‘cycling rate’ or effective frequency of the response (this is often conservatively taken as the natural frequency of the structure, $f_1$) and $T$ is the time interval over which the maximum value is required, in seconds. Wind tunnel studies often factor the
ISO criteria by 0.7 to 0.8 for a residential tower.

The Council on Tall Buildings and Urban Habitat (CTBUH) have recently suggested that the torsional velocity should be below $3\, (\text{milli-rad})/s$ for a 10 year return period wind load (Isyumov and Tschanz 2001). These are tentative guidelines and have not yet been included in any wind codes. It has been suggested that torsional motions may be more critical to occupant comfort than horizontal motions (Simiu and Miyata 2006).

2.4.2 Ultimate Limit State Design Strategy

2.4.2.1 Strength Design of Coupling Beams with Non-Diagonal Reinforcing Details

The strength of coupling beams in high-rise buildings in non-seismic regions is typically governed by shear. This is due to the relatively short clear spans that result in large shear forces when the system is subjected to lateral loading. Diagonal reinforcing details are typically not used in non-seismic regions primarily because the lintel beam heights are minimized for architectural reasons and thus cannot accommodate this reinforcement layout. The Design of Concrete Structures, CSA A23.3-04 (CSA 2004) outlines a design method based on the “modified compression field theory” (MCFT) (Vecchio and Collins 1986) for the strength design of RC coupling beams. At the preliminary lateral design stage of a project, the maximum shear capacity of the cross-section, $V_{r,\text{max}}$, must be met to ensure there are no problems with the beam size and lateral load resisting system at the final strength design (Clause 11.3.3):

$$V_f < V_{r,\text{max}} = 0.25 \phi_c f'_c b_w d_v$$

(2.47)

where $V_f$ is the factored shear force, $\phi_c$ is the resistance factor for concrete (0.65), $f'_c$ is the specified concrete strength, $b_w$ is the effective minimum effective web width. The effective shear depth, $d_v$, is taken as the greater of $0.9d$ and $0.72h$, where $d$ is the distance from the extreme compression fibre to the centroid of the longitudinal reinforcement and $h$ is the overall depth of the beam.

The forces in each lintel beam are determined from a FE analysis using the factored 1 in 50 year wind storm loading. If Eq. (2.47) is not met, often times engineers will further refine the stiffness distribution to account for the additional cracking caused by higher stress using the same iterative process, until Eq. (2.47) is satisfied. If this cannot be achieved a major redesign of the lateral load resisting system might be necessary. This may affect the economic feasibility of the entire project, because most of the unit layouts are finalized if not already sold to future tenants at the final design stage.

After the factored design forces are determined, the reinforcement of the beams is designed using the CSA A23.3-04 for flexure (Section 10) and shear (Section 11).
2.4.3 Design of Walls

The design of the walls is somewhat simpler than that of the coupling beams design as the walls typically behave predominantly in flexure and CSA A23.3-04 (CSA 2004) outlines a simple procedure for the design of such members. One important consideration can be the tensile forces in the leeward walls under the factored wind loads. If the walls large gravity compressive forces are overcome by the coupling tensile forces induced by the wind, then the coupled wall system may have significant flaws and major overhauls in the design may be required.

2.4.4 Design Procedure

2.4.4.1 Wind Design of Coupled Wall High-Rise Buildings

Structural high-rise designers assume that the buildings remain linear elastic under wind loads and use 3-D FE modelling software programs to design the structural members and check deflections. A description of the types of models that are used were described earlier in Section 2.2.1.3. Typically, walls are modelled using shell elements while beams are modelled using frame elements. There are two models used for the analysis: i) an SLS model with representative effective stiffness properties of members plus the 1 in 10 year wind loads, and ii) an ULS model with representative effective stiffness properties of members plus the 1 in 50 year wind loads. The dynamic building response is determined by a wind tunnel study based on modal analysis properties obtained from the structural engineers’ models. The dynamic response of the structure is converted into inertia loads distributed throughout the structure and the wind loads are given to the structural engineer as a total load (addition of static and dynamic inertia loads) to be applied at all diaphragm locations. Typically 24 load combinations are used to account for all possible wind effects on the building. The structural designers use these load combinations to create a maximum ULS design envelope for each member of the building. Each member is then individually designed.

There are three performance criteria that must be met for the lateral design of a high-rise building:

1) SLS Human Comfort Criteria (Across-wind and Torsional Velocity Limits),

2) SLS Drift Criteria,

3) ULS Strength Criteria of all structural members,
A suggested design flowchart for wind design is shown in Figure 2.20. The steps of the high-rise design are outlined below:

**STEP 1)** Establish the functional layout of the building with the architect and other sub-consultants. The floor-to-ceiling heights, concrete core location and wall locations establish the unit divisions.

**STEP 2)** A preliminary lateral load resisting system is established. The structural engineer designs the concrete walls and coupling beams. This includes the wall widths and lengths, coupling beam depths, reinforced concrete strength and preliminary reinforcing profile up the height of the building.

**STEP 3)** Using a 3-D FE model of the building, a SLS and ULS lateral analysis with the appropriate stiffness assumptions for the concrete elements and effective wind loads are carried out.

**STEP 4)** The storey drift limits are checked based on the SLS wind loads.

**STEP 5)** The ULS factored wind loads are used to check the ultimate strength limits, focusing on coupling beams and the wall elements.

**STEP 6)** A wind tunnel study is conducted using the SLS modal properties obtained from the structural engineer. The acceleration and torsional velocity response is obtained from the wind tunnel study and new loads and load combinations are obtained.

**STEP 7)** Steps 3 to 5 are conducted again with the new wind tunnel loads.

If any of the design requirements are not met a new lateral load resisting configuration must be attempted. In general, the structural engineer will be attempting to make the structure as stiff as possible without compromising the architectural functionality of the building. At some point after many iterations of optimizing the member sizes and structural layout with design architects, the structural design team may inform the owner that a major overhaul of the lateral load resisting system, or the inclusion of a supplemental damping system, is necessary.

### 2.5 SEISMIC DESIGN PHILOSOPHY

#### 2.5.1 Seismic Behaviour of Coupled Wall Buildings

Under severe earthquakes tall RC coupled wall buildings are expected to yield. Figure 2.21a shows the structural members that are expected to yield in light red. The nonlinear response of coupling beams and the concrete walls at their base are intended to form the global ductile mechanism. All other members in the structure are designed to remain linear elastic when the ductile yielding mechanisms are undergoing their large nonlinear deformations.
Figure 2.20 Design Approach for RC High-Rise Coupled Wall Buildings under Wind Loading
Figure 2.21b shows some photos of coupling beams in Chile after the Conception Earthquake 2010 (LATBSC 2010). The pictures show strong evidence of coupling action between walls. Figure 2.21c shows some photos of shear walls at the base of the structure. The photos were taken from high-rise condominiums constructed in the past 25 years. The condominiums in general behaved as expected during the earthquake, however the condo owners were extremely dissatisfied with their structures’ performance, with many of the owners demanding demolishing the building and construction of a new condominium (LATBSC 2010).

2.5.2 Review of Research on Cyclic Behaviour of Coupling Beams

Christopoulos, Bentz and Collins (2003) studied the inherent damping characteristics of RC coupled beams analytically. Specifically, refined nonlinear analyses of the beams were conducted to determine if low level cracking contributed to the inherent damping of the structure. The study concluded that little cyclic energy dissipation could be relied upon by the low level cracking of the lintel beams.
An experimental study by Morawski (1980) was conducted to determine if coupling beam shear capacity is affected by cyclic loading. Four identical specimens were tested at the University of Toronto, Structural Testing Facility. Figure 2.22a shows the tested specimens and Figure 2.22c shows the test setup. The experimental test results are shown in Figure 2.22b for all four test specimens. The conclusion of the study was that the repeated load reversals had an insignificant effect on the shear behaviour and that the shear capacity response can be predicted adequately by monotonic loading procedures provided that the longitudinal reinforcement does not extensively yield.

![Coupling Beam Specimen](image1)

**a) Coupling Beam Specimen**

![Experimental Results](image2)

**b) Experimental Results**

![Test Set-up](image3)

**c) Test Set-up**

Figure 2.22 Effect of Load Reversals on Coupling Beam Shear Capacity (after Morawski 1980)

In earthquake zones, diagonally reinforced coupling beams have been shown to be very effective in achieving a ductile response with little strength degradation (Paulay 1969 and Paulay and Priestley 1992). They consist of diagonally reinforcing bars placed from one corner of the coupling beam to the other. To avoid lateral buckling at large nonlinear load reversals, the diagonal bars are supported laterally with closely spaced stirrups. They have been used in many buildings in seismic regions and a common detail is shown in Figure 2.23a (ACI 2004 and Wallace 2006). Unfortunately, the details provided in the code are difficult to build and require significant labour. In some projects it was estimated that the diagonal coupling beams required an additional day of construction per floor (Wallace 2010). Recently there have been new details developed to address some of these constructability issues (Naish et al. 2009 and Canoblat et al. 2005), seen in Figure 2.23b and 2.23c. The test results of one of the new more constructible beams is shown in Figure 2.23d.
2.5.3 Design Guidelines

Recent guidelines have suggested some best-practice principles for the design of high-rise buildings for different seismic hazard levels (SEAONC 2007, CBTUH 2008, LATBSDC 2008 and PEER 2009). These procedures and guidelines were developed because designers of high-rise buildings in seismic regions found that the building codes, such as the International Building Code (ICC 2003), were too restrictive and were more suitable for low- to mid-rise buildings. To ensure good practices are followed, peer review expert panels usually review the design of high-rise buildings. Typically, the peer review panel includes experts in high-rise building design, performance-based earthquake engineering, nonlinear dynamic analysis, inelastic behavior of various materials, components and systems, geotechnical engineering and seismic hazard.
analysis. A summary of some of the main elements of these guidelines are provided below. These guidelines serve as an overall road map for the procedure to follow while the finer details would be established with the peer review panel on each individual project.

The most critical requirement of the guidelines is that capacity design principles are applied and that there are suitable ductile yielding mechanisms under large nonlinear lateral deformations. Another aspect of these guidelines is that they recommend using more conservative values of damping that are more representative of the inherent damping properties that have recently been measured on similar buildings, and any additional damping is assumed to come from the excursion into the nonlinear range of structural members and is explicitly accounted for through the nonlinear modelling of the individual members. For example, the CTBUH recommends using damping levels of 1% to 2% for buildings of height 50m to 250m. This is more reflective of the building behaviour compared to other design codes that assume a damping of 5% for all modes of vibration. Also, it is suggested that such tall buildings be monitored to assess dynamic properties over the life of the structure.

The earthquake ground motions are typically seven earthquake time-histories, scaled to the Service Level Earthquake (SLE) and the Maximum Credible Earthquake (MCE) response spectrums, respectively. Each performance measure is evaluated based on the average response of the seven records. There are three levels of earthquake ground motion that are considered as defined in ASCE 7-05 (ASCE 2006) and the high-rise building Performance-Based Seismic Design Guidelines (SEAONC 2007, CBTUH 2008, LATBSDC 2008 and PEER 2009). The characteristics for the i) Service Level Earthquake, ii) Design Level Earthquake and iii) Maximum Credible Earthquake are described below:

1) **Service Level Earthquake (SLE)** - Negligible damage under the SLE. The SLS is a once-in-a-lifetime earthquake with approximately a 50 year return period. This is demonstrated by ensuring almost all building elements and nonstructural components remain essentially linear elastic. This is assessed using a 3-D linear or nonlinear analysis.

2) **Design Level Earthquake (DLE)** - The design level earthquake is defined as an earthquake with an intensity represented by an elastic response spectrum that has 2/3 of the amplitude of the MCE. Adequate performance is assessed using 3-D nonlinear analysis. “Fuse” mechanisms have been activated and all non-ductile members must have force demands less than the nominal strengths to demonstrate adequate performance.

3) **Maximum Credible Earthquake (MCE)** - The structure needs to demonstrate a low probability of collapse under the MCE. The MCE is an extremely rare earthquake with approximately a 2500 year return period. Collapse prevention is achieved by demonstrating all “fuse” mechanisms have adequate deformation capacity and all force demands in elements with non-ductile failure modes are less than the nominal strengths. This is assessed by 3-D nonlinear analysis.
2.5.4 Design Procedure

A suggested design flowchart is shown in Figure 2.24 and summarized below:

STEP 1) Establish the functional layout of the building with the architect and other sub-consultants. The floor-to-ceiling heights, concrete core location and wall locations establishing the units division. The architect is informed of the “fuse” mechanism and sufficient depth is made available for the coupling beams.

STEP 2) A preliminary lateral load resisting system is designed with the structural engineer. The structural engineer designs the concrete walls and coupling beams (wall widths and lengths, coupling beam depths, reinforced concrete strength and preliminary reinforcing profile up the height of the building). “Fuse” elements’ strength and rotation capacities are estimated based on the reinforcing profiles.

STEP 3) Using commercial software the SLE, DLE and MCE 3-D FEM lateral analysis models including are created. Scaled earthquake records are created based on the site specific SLE and MCE response spectra.

STEP 4) The SLE drift and acceleration limits of the nonstructural building components are checked against the established performance values. The structural members are also checked to ensure they have not reached their member strength.

STEP 5) The DLE deformation capacity and strength degradation is checked for walls and coupling beams. The strength limits of all other members are checked along with the deformation limits of nonstructural components.

STEP 6) The MCE deformation capacity and strength degradation is checked for walls and coupling beams. The strength limits of all other members are checked along with the deformation limits of nonstructural components. and that there is a low probability of collapse.

STEP 7) The design is reviewed by the peer review panel.

If any of the design requirements are not meet the design team must reevaluate the lateral load resisting scheme for the building.
Fork Configuration Dampers (FCDs) for Enhanced Dynamic Performance of High-Rise Buildings

### CHAPTER 2: Behaviour and Design of RC Coupled Wall High-Rise Buildings

Step 1: Establish functional layout with architect and owner with Capacity Design Approach
- Establish lateral load resisting system with well defined inelastic behaviour in beam and wall elements
- All other members designed to remain elastic during earthquakes

Step 2: Geometric and structural layout
- Establish wall lengths and widths
- Establish coupling beam depths
- Establish concrete strengths
- Design coupling beam and wall reinforcement

Step 3: Create SLE, DLE and MCE 3-D FEM lateral analysis

Step 4: Check SLE response using linear time-history analysis
- Establish SLE stiffness distribution
- Check essentially elastic response (negligible damage) to structural and nonstructural members
- Check that no appreciable permanent deformations are sustained
  - Check drift limits
  - Check acceleration limits

**NO** If SLE requirements not met go to Step 2

**YES**

Step 5: Check DLE response using nonlinear time-history analysis
- Establish DLE stiffness distribution
- Check deformation limits of coupling beams and walls
- Check strength degradation of coupling beams and walls
- Check strength limits of other members

**NO** If DLE requirements not met go to Step 2

**YES**

Step 6: Check MCE response using nonlinear time-history analysis
- Establish MCE stiffness distribution
- Check deformation limits of coupling beams and walls
- Check strength degradation of coupling beams and walls
- Check strength limits of other members
- Check low probability of collapse

**NO** If MCE requirements not met go to Step 2

**YES**

Step 7: Submit report to seismic peer review panel
- Reviews structural aspects and advises building officials

**NO** If seismic peer review panel find shortcomings go to Step 2

**YES**

Lateral Seismic Design Complete

*Figure 2.24 Design Approach for RC High-Rise Coupled Wall Buildings under Earthquake Loading*
2.6 CURRENT METHODS TO MITIGATE LATERAL VIBRATIONS

An aspect that may be not well accounted for in the wind design of high-rise buildings has been brought out recently in performance-based seismic engineering techniques applied to RC structures subject to various levels of wind loads. The low damping values associated with high-rise buildings have led to the development of supplemental damping devices or vibration absorbers that may be added to the structure. Vibration absorbers are focused on in this chapter since they are the most common systems used for high-rise buildings.

2.6.1 Passive Dynamic Vibration Absorbers

Dynamic Vibration Absorbers are currently the most common method to mitigate wind vibrations in high-rise buildings and have recently been considered for seismic protection of structures. Dynamic Vibration Absorbers transfer some of the structural vibrational energy from the primary structure to the absorber. They are installed at the top of high-rise buildings and are calibrated to be in resonance with a specified mode of vibration of the structure (typically the first). They can be thought of conceptually as a force whose magnitude is dependent on the effective mass inertia of the vibration absorber that is 180° out of phase with the building response. There are three types of the Passive Dynamic Vibration Absorbers: i) Tuned Mass Dampers (TMDs) and Tuned Liquid Dampers which include ii) Tuned Sloshing Dampers (TSDs) and iii) Tuned Liquid Column Dampers (TLCDs).

2.6.1.1 Tuned Mass Dampers (TMDs)

Tuned Mass Dampers have been studied for structures as early as 1909 by Frahm (Den Hartog 1956). Frahm’s absorber, shown in Figure 2.25, consists of a relatively small mass, \( m \), and spring with stiffness, \( k \), attached to the system’s main mass, \( M \), with the spring stiffness, \( K \).

Subject to a simple harmonic load, \( F(t) = F_0 \sin \omega t \), one can show that the mass, \( M \), can be kept stationary when the natural circular frequency of the attached absorber, \( \omega_a = \sqrt{\frac{k}{m}} \), is tuned to the actual excitation frequency, \( \omega \).
TMD theory was developed by Den Hartog (1956) for the SDOF Main Structure - TMD system displayed in Figure 2.26, consisting of a TMD with mass, \( m \), stiffness, \( k \), and damping, \( c \), attached to the Primary Structure with mass, \( M \), stiffness, \( K \), and damping, \( C \), with a frequency, \( \omega_s = \sqrt{\frac{K}{M}} \). Den Hartog provided the solution for the system with \( C = 0 \), subject to harmonic loading \( F(t) = F_0 \sin \omega t \), and developed optimum tuning parameters. The theory was later extended to the case where the damping of the main structure is not zero, \( C \neq 0 \), and \( F(t) \) is a random vibration load subject to a constant spectral density or white noise (Crandall and Mark 1963). TMDs are typically designed to minimize the primary structural system response, and for various excitations there are different optimization criteria. Warburton (1982), performed a thorough analysis to determine the optimum vibration absorber parameters with different optimization criteria for different loadings of the SDOF Primary Structure without damping.

![Figure 2.26 Damped Main Structure and TMD System](image)

In 1977, McNamara demonstrated that TMDs are effective in reducing wind-induced structural vibrations of high-rise buildings (McNamara 1977). The optimum parameters for TMDs subject to seismic excitations was provided by Sadek et al. (1997).

Christopoulos and Filiatrault (2006) have a summary of many approaches and optimum tuning conditions for TMDs and in addition also developed optimum tuning conditions for undamped and damped buildings under seismic base excitation.

Developers of TMDs for wind vibrations view the additional TMD as changing the damping ratio in the primary structure, \( \xi_s = C/(2\sqrt{KM}) \), to a larger effective damping, \( \xi_{eff} \), of the damped primary structure and TMD system. The theory comes largely from using results from McNamara (1977) and Crandell and Mark (1963).

Luft (1979) estimated the effective damping of a high-rise building with a TMD, subject to random vibrations in one lateral direction and minimizing the top floor accelerations, \( \xi_{eff} \):

\[
\xi_{eff} \approx 0.95 \sqrt{\xi_s}/4 + 0.35 \xi_s \tag{2.48}
\]
where, \( \mu = \frac{m}{M} \), is the mass ratio.

In the complex Multi-Degree of Freedom Response (MDOF) dynamic response of high-rise buildings, where the frequency of the primary structure changes over time and the structure is subject to many different types of environmental excitations such as wind and earthquake loading with a broad band of frequencies, the performance is not expected to be the same as the one discussed above (Soong and Dargush 1997 and Christopoulos and Filiatrault 2006). Carr (2005) studied the seismic response of buildings with TMDs, where the structural members underwent nonlinear deformations. In the study it was shown that as structures undergo nonlinear deformations there is a detuning effect which causes a loss of efficiency of the TMD and the building response can sometimes even be amplified.

### 2.6.1.1.1 Applications of TMDs and Reliability Issues

Much of the early development of TMDs was directed towards mechanical systems, where the frequency of the TMD was tuned to the operating frequency or excitation frequency of the machine or primary structure, and only more recently have they been applied to mitigate wind and seismic vibrations. For civil structures, the TMDs consist of heavy masses, usually on the order of 2% to 4% of the total mass of the primary structure. These are located at the top of the structure, and they tend to remain still as the structure beneath moves, while transferring inertia forces to the structure that opposes the structural motion. The first two applications of TMDs in civil engineering structures were the Centerpoint Tower in Sydney, Australia and the CN Tower in Toronto. The first high-rise applications were the Citicorp Center in New York (McNamara 1977), displayed in Figure 2.27a and the John Hancock Tower in Boston (Wiesner 1979), displayed in Figure 2.27b, where dual TMDs were applied to control both torsional and lateral motions.

The TMD used in the Citicorp building consists of a \( 10 \times 10 \times 3 \) m concrete block weighing 410 tons located at the top floor of the building. There are two nitrogen-charged pneumatic springs and two active hydraulic actuators. The spring stiffnesses are tuned by changing the pneumatic pressure and operate in both lateral directions with the capability of 45 inches of displacement travel. Accelerometers are used to activate the TMD when the top storey lateral accelerations exceed 3 milli-g and the TMD is operational until half an hour after the motion decreases below 0.75 milli-g. Additionally there is an anti-yaw device to prevent the concrete block from twisting and snubbers to prevent excessive motion. The concrete mass is supported on twelve pressure-balanced bearings controlled by a hydraulic pump.

The TMD system installed in the John Hancock building was not included in the original design and was installed later in order to mitigate occupant discomfort. Due to the shape and vibration properties of the building, a TMD system was needed for damping out vibrations in the short direction and also in torsion. In this application there were two 300 ton lead blocks with a 74 m lever arm to resist torsion,
Fork Configuration Dampers (FCDs) for Enhanced Dynamic Performance of High-Rise Buildings

CHAPTER 2: Behaviour and Design of RC Coupled Wall High-Rise Buildings

- Fork Configuration Dampers (FCDs) for Enhanced Dynamic Performance of High-Rise Buildings

Activating at a lateral acceleration of 3 milli-g. They are supported on sixteen pressure-balanced bearings controlled by a hydraulic pump. Kijewski and Kareem (1999) have provided data from monitoring of the building during the first 5 years of occupation. The damping ratio data showed significant scatter, varying from 1.95-3.07%.

Figure 2.27 TMD Applications

- John Hancock Building, Boston (adapted from Soong and Dargush 1998)
- Taipei 101, Taipei (RWDI 2006)

a) Citicorp Building, New York (McNamara 1977)
Both the John Hancock and Citicorp TMDs are theoretically passive devices, but they both require power to levitate the masses. There is also concern that during large windstorms, the TMDs may lose power when they are most needed. Also, TMDs require a combination of many mechanical and electrical systems to perform effectively in addition to the simple mass, damper and spring system.

Pendulum type TMDs, as seen in the Taipei 101 building in Figure 2.27c, have been designed to accommodate these constraints. The Taipei 101 vibration absorber consists of 41 steel plates, weighing 662 tonnes and extends over 5 stories. The TMD frequency is based on the length of the suspension system and the mass that is suspended. The pendulum type TMDs are typically augmented with coil springs for fine tuning.

Passive TMDs have not made much of an impact in the seismic protection of high-rise buildings, because of the de-tuning effect when high-rise buildings yield under seismic displacements and because they only target one mode of vibration. Active and semi-active mass dampers have recently been used in Japan for high-rise buildings for both wind and earthquake loading (Constantinou et al. 1998). Active and semi-active damper systems develop control forces based on information from sensors and a control algorithm applying forces generated from an external power supply. The active and semi-active dampers are complex and due to their lack of use for mitigating wind vibrations will not be discussed further. Further information can be found in Fujita (1993), Higashino and Aizawa (1993), Nakamura et al. (1996), Housner et al. (1997), Constantinou et al. (1998), Kareem et al. (1999).

TMDs have generally been used for steel buildings where much more accurate predictions of the natural frequencies can be achieved with FE models. As indicated in Section 2.2.3, for concrete buildings, sometimes these predictions can be off by as much as 50% and will change over the life of the structure. Also, TMDs are most commonly used to solve lateral vibration problems with little torsional vibration component. Typical layouts for coupled wall structures are not very torsionally stiff and often times have modes of vibration that are coupled in all directions (lateral X-sway and Y-sway and torsion). Therefore the applicability to large concrete coupled shear wall structures can be questioned. Since TMDs typically only target one lateral mode of vibration they are not useful for the many other higher modes, which can contribute up to 10-20% of the total wind-induced accelerations, and TMDs can even amplify motion in some of the higher modes of vibration. If the high-rise structure is subject to an earthquake, the device is usually designed to lock up to avoid excessive damper displacements (Christopoulos and Filiatrault 2006 and Carr 2005). If this occurs, the TMD is then merely an added mass at the top of the structure, thus increasing the inertia forces.
The use of TMDs in high-rise buildings is often dictated by practical and economical conditions for the client, such as: i) the initial capital cost, ii) the loss of expensive floor space (the top stories in high-rises are typically the most sought after property) and iii) the loss of floor space due to increased members sizes to support the TMDs, iv) operating costs and v) maintenance and safety costs.

2.6.2 Tuned Liquid Dampers: Tuned Sloshing Dampers (TSDs) and Tuned Liquid Column Dampers (TLCDs)

Tuned Liquid Dampers are relatively new devices for building and structural applications, but they have been used in marine and aerospace structures for many years. Both TSDs and TLCDs have recently begun to gain popularity and are becoming increasingly used in high-rise structures to control wind-induced vibrations. TSDs and TLCDs are similar in principle to TMDs with a heavily-damped auxiliary system at the top stories of high-rise buildings, however the mass, damping and stiffness are provided by motion of the moving liquids, as illustrated in Figure 2.28.

![Figure 2.28 Tuned Liquid Damper Systems](image)

Tuned Sloshing Dampers consist of a large rigid tanks, filled with a shallow liquid (typically water), absorbing and dissipating lateral vibrational energy when it sloshes. The tank is designed such that the frequency of the free surface wave is close to the fundamental frequency of the primary structure. The frequency of the free surface wave is determined by the length and depth of the liquid. TSDs began to be examined for the reduction of wind-induced responses of high-rise buildings in the late 1980s (Kareem and Sun 1987 and Modi and Welt 1988). Although TSDs are simple in concept, the physical mechanism of the sloshing water is very complicated and the response of TSDs is highly nonlinear. Parametric studies on the behavior of circular TSDs subject to free vibrations, with height \( h \) of the liquid and radius \( R \) of the tank, were carried out by Fujino et al. (1988). It was concluded that the additional damping is highly dependent on the amplitude of vibration and the fundamental frequency of the absorber can be expressed as:
For simplicity, TSDs are modelled as equivalent linear systems using well-known TMD theory (Kareem and Sun 1987). One advantage of TSDs over other types of vibration absorbers is that studies have shown that both rectangular (Housner et al. 1997, Tait 2005 and Tait et al. 2007) or circular tanks (Fujino et al. 1988) may be applicable for controlling vibrations in both principle directions. Damping screens or breaking columns are often used to increase the energy dissipation capacity of the TSD (Noji et al. 1989 and Tait 2005).

Tuned Liquid Column Dampers consist of a column of liquid moving in a U-shaped tube, with energy dissipation being achieved using orifices or screens at the base of the U that control the fluid moving from one column to another, with restoring forces provided by gravity (Sakai et al. 1989, Hitchcock et al. 1997a and 1997b). When a large dynamic load is applied to the high-rise building, the inertia of the water causes the water to oscillate through the passage between the two columns. The TLCD has its own natural frequency, which is determined from the tank geometry, and when the period is close to the period of the primary structure, the water motion becomes significant, transferring the vibrational energy to the water. TLCDs have been examined for applications in wind engineering by Xu et al. (1992) and for seismic applications by Won et al. (1996) and Sadek et al. (1997). Saoka (1988) determined a simple approximate formula representing the frequency of TLCD with length of liquid in the columns $L$, and verified experimentally by Sakai et al. (1989):

$$\omega_n = \frac{1.84g \tanh \left( \frac{1.84g}{R} \right)}{R}$$

(2.49)

where $g$ is the acceleration of gravity.

Passive TLCDs have dramatically different performances depending on the dynamic loading characteristics and they are designed and tuned for the worst design case loading conditions but are non-optimal at other loading conditions. Kareem (1994) and Haroun (1994) suggested to use either active and semi-active orifices to control the amount and rate at which water flows from column to column. Soong and Dargush (1997) provided a detailed review of Tuned Liquid Dampers.
2.6.2.1 Applications of TLDs and Reliability Issues

TSDs were first used by Frahm in 1902, for the rolling motions of large ships (Den Hartog 1956) and later were applied to reduce vibrations of space satellites (Carrier and Miles 1960 and Bhuta and Koval 1966). Shallow water TSDs have been applied in many tall tower structures subject to wind-induced vibrations, primarily in Japan (Wakahara et al. 1992 and Tamura 1995). The first two TSD applications were in Japan in 1987 at the Nagasaki Airport Tower and the Yokohama Marine Tower (Tamura et al. 1995). The first high-rise building structure to implement TSDs was the 150m tall Shin Yokohama Prince Hotel. In this project the engineers positioned 30 multilevel circular vessels with external diameters of 2.14m and heights of 2.15m around the periphery of the upper floor. The TSDs liquid mass was 1% of the total mass of the building and were tuned to a sloshing frequency of 0.31 Hz. A recent example of a TSD incorporated into a 60-storey high-rise structure in New York City was described in Robinson et al. (2007). A 1:9 scaled model of the TSD was built and tested to estimate the TSD behaviour. The inner dimensions of the TSD are 13.7m by 5.5m with a mass of 190 tons. Large cast-in-place concrete walls with a waterproof membrane and sensors monitoring the water levels to predict the natural frequencies were provided.

The first practical applications of TLCDs dates back to the 1880s when Watts (1883) introduced them to prevent motion of shipping vessels. The first TLCD applications for wind sensitive civil structures occurred in Japan (Tamura et al. 1995) and the first TLCD used in a high-rise building in North America occurred at the Wall Center in Vancouver Canada (Glotman Simpson 2006). The Wall Center is a 48-storey residential RC coupled core building with three outriggers connecting the core to exterior columns. The wind tunnel predicted accelerations of 40 milli-g, because the structure geometry makes it sensitive to across-wind vibrations. There were two identical 4-storey high tanks, containing 50,000 gallons (600 tons) of water each. Multiple TLCDs with adjustable sluice gates have been used in the One King West building in Toronto. The sluice gates are adjustable for future timing.

Compared to TMDs, the operation of TSDs and TLCDs require minimal power to control the sluice gates or control valves. A single TLCD is only effective in one direction, but by combining two TLCDs offset by a distance, they may also work in torsion. Rectangular or circular TSDs have an advantage over other vibration absorbers, because they can be tuned to both principal directions, but have not been used effectively for torsional motions. Due to the fact they both have controlling mechanisms, they can adapt to the changing frequency of the structure. The response of complex multi degree of freedom buildings with TSDs systems to different excitations such as earthquakes, and more research is needed to investigate this. The water may be used to service the entire structure, but they are generally very large and take up multiple stories over the height of the building in the most sought after space.
2.7 COMPARISON OF WIND DESIGN AND SEISMIC DESIGN

Performance-based seismic engineering has offered a more rational approach to gain insight into the redundancy of buildings subjected to large and rare earthquakes. Lately, there has been some thoughts about applying this concept to buildings under wind loads (Bracci 2008). As large design based wind storms become more frequent and unpredictable due to climate change (Steenbergen et al. 2009, Kareem 1988), the desired wind-performance subject to rarer wind storms becomes an important concern. Also, the upstream terrain can change significantly over the service life of the building causing the air flow to become much more turbulent than that the tower was not originally designed for. A pushover curve of a high-rise building and the expected performance under wind and earthquake loads are used to illustrate the building behaviour in Figure 2.29.

As described in Section 2.5, when buildings are subjected to large cyclic seismic deformations, “fuse” elements are designed using a capacity design methodology concentrating nonlinear action in specific regions. These elements are distributed throughout the height of the structure and offer a redundant and safe design for many levels of earthquake excitation. Buildings in seismic zones have stringent seismic detailing for structural members satisfying a large range of properties.

On the other hand, as described in Section 2.4, structural members are designed to remain essentially elastic or just below the incipient yield point at the 1 in 50 year wind load. Beyond the design basis wind loads (for example a 1 in 500 year wind load) the structure response becomes much more uncertain. The member design is based on scaled up design based wind loads using factors of safety, but the forces are based on the predicted wind speed, stiffness and damping estimates of the building. Since wind storms are a much more frequent events than earthquakes, there are questions about the fatigue response of members and factors of safety with multiple excursions into the nonlinear range.
Tuned Mass Dampers (TMDs) and Tuned Liquid Dampers (TLDs) as discussed in Section 2.6 are optimized for certain design conditions. Recent studies have optimized the performance of the system for the 1 in 50 year wind storm and have taken advantage of the added damping to reduce the overall wind loads of the structure and the members (Robinson et al. 2008). If there is a rare wind event that occurs that is larger than the design basis wind storm the concrete elements would crack much more significantly than expected and the TMD or TLD would be tuned to an incorrect frequency. This would lead to much more uncertainty under rare events and dependability of the TMDs.

2.8 CONCLUSIONS

Coupled wall buildings were introduced in Section 2.2 and the behaviour of high-rise buildings subjected to wind was described in Section 2.3. Some aspects of the high-rise design process as well as the behaviour of high-rise buildings subject to wind and seismic loads was described in Section 2.4 and Section 2.5. Section 2.6 describes vibration absorbers and Section 2.7 provided a general comparison between wind loading and seismic loading. Considering these design challenges there is need for a reliable damping system to provide distributed viscous damping to assist with the design of high-rise buildings. The goal of this thesis is to study damping systems, develop a new system which could be applied, and solve some of the particular design challenges that exist and to test and validate the system.
CHAPTER 3:  FORK CONFIGURATION DAMPERS (FCDs) FOR USE IN RC COUPLED WALL HIGH-RISE BUILDINGS

3.1 CURRENT CHALLENGES IN HIGH-RISE DESIGN AND NEED FOR ADDED DAMPING

The current practice for high-rise reinforced concrete buildings were summarized in the previous chapter. The dynamic behaviour of high-rise buildings has become a critical design consideration, as buildings are built taller and more slender. Wind vibrations cause an increase in the design lateral wind loads, but more importantly, they can be perceived by occupants of the building and can create levels of discomfort ranging from minor annoyance to severe motion sickness. The techniques used in design to address these challenges include reducing the number of stories, stiffening the building, or incorporating a vibration absorber at the building top. For concrete high-rise buildings the most common strategy is to stiffen the structure. This is done by increasing the size of the concrete members and adding additional lateral load resisting systems such as concrete outriggers. This has many economic consequences including the cost of the construction materials and construction time, plus it reduces the usable space in the building. For high-rise buildings, stiffening the building is sometimes insufficient to mitigate the vibration problems and therefore the number of stories must be reduced or vibration absorbers must be installed.

Vibration absorbers consist of large tuned liquid or tuned mass dampers (TLDs and TMDs) that are complex to design and typically occupy the entire top floor of the building, which is the most valuable space. In addition, for such a damper to be effective, a substantial additional mass, usually of the order of 2% to 4% of the mass of the entire building, must be positioned at the top floor, which results in an increase of gravity loads that must be carried over the entire height of the building. TLDs and TMDs can only be tuned to a single mode of vibration and lose their effectiveness as the vibration absorber’s frequency diverges from the actual building frequency. Concrete buildings have been shown to have different natural periods over the life of the building and are difficult to estimate using current modelling techniques, raising questions on the performance of such systems if not properly tuned. Also, these vibration absorbers are not effective for earthquakes, since the building response to earthquakes is a function of many modes of vibration.
During earthquakes, tall buildings can sustain large amounts of damage to structural and nonstructural members. For concrete coupled wall buildings, coupling beams and walls with stringent ductile reinforcing detailing requirements act as “fuse” elements during large earthquakes. The coupling beams often use diagonal reinforcing patterns and have constructibility issues adding to the overall construction time and cost of a project. Also, questions arise about how to economically repair a building after damage has occurred during an earthquake.

Recent in-situ measurements on the dynamic properties of high-rise buildings have shown that damping decreases significantly with building height and is lower than typical values assumed during design. The values assumed in design (often 2% of critical in the first few modes) are unconservative and measurements have been made on actual buildings showing damping levels of less than 1% of critical. If these actual measured damping values were used in wind-tunnel techniques at the design stage, there would be an increase in design challenges and a greater need for the use of damping systems.

Passive dampers configured in brace applications have been shown effective in reducing vibrations in low-rise buildings, but have not been shown to be particularly effective for high-rise buildings. This is because when subject to lateral motion, high-rise buildings deform in a cantilever-like shape, as opposed to typical brace configurations that deform in an interstorey shear configuration and therefore can not effectively engage the dampers positioned in braced configurations.

Considering these current challenges in designing high-rise structures, there is need for a robust and reliable distributed damping system that adds linear viscous damping throughout the height of buildings. The Viscoelastic (VE) “Fork Configuration Damper” (FCD, patents pending in 7 countries) was invented and developed at the University of Toronto to address these challenges.

The goals of the FCD are outlined below:

1) Add distributed linear viscous damping in RC coupled wall high-rise buildings without altering the fundamental period of the structure significantly,

2) The damper must be cost-effective and non-obtrusive, fitting within current lateral load resisting systems,

3) The damper must be practial,

4) The damping system must be able to damp out vibrations effectively over the expected design life of the structure, including both wind vibrations and earthquake vibrations. Also, the damper must be capacity designed to ensure ductile behaviour under extreme loads.
The FCD concept is introduced in this chapter. Chapter 4 presents the results of small-scale VE material tests conducted at the University of Toronto to establish VE material characteristics. It also presents full-scale FCD tests conducted on two-FCDs, one designed for an actual high-rise building application in downtown Toronto (FCD-A) and one designed with a structural “fuse”, for applications in high seismic areas (FCD-B). Chapter 5 introduces modelling techniques for the FCDs in high-rise buildings. Finally in Chapter 6, design recommendations for wind and seismic applications are presented, followed by a case study of a high-rise building designed for downtown Toronto are presented.

### 3.2 VE FORK CONFIGURATION DAMPERS (FCDs)

In coupled wall high-rise buildings (Figure 3.1a) the primary lateral load resistance is provided by large RC walls that are connected together with RC coupling beams as shown in in Figures 3.1b and 3.1c. The coupling beams increase the stiffness of the lateral load resisting system, as discussed in the previous chapter. When a traditional coupled wall building deflects due to the applied lateral wind or earthquake loads the large walls bend about their centerlines and cause the coupling beams to rotate and displace vertically, bending in double curvature. The deformed shape of coupled walls subject to lateral loads is illustrated in Figure 3.1d. In this structural configuration the coupling beams undergo large shear deformations.

![Figure 3.1 Coupled Wall Buildings](image)

The proposed FCDs are introduced in lieu of RC coupling lintel beams between the walls to take advantage of these large shear deformations. Figure 3.2a illustrates a potential FCD configuration in a high-rise coupled wall structure and Figure 3.2b shows a plan view of the damper itself. FCDs utilize multiple layers of VE material sandwiched between and bonded to multiple steel plates. The VE alternates between layers of steel plates with each consecutive steel layer extending out to the opposite side. For the damper shown in Figure 3.2b all steel plates are bolted together in slip-critical connections with built-up I-section...
steel anchorages. Each set of steel plates resembles a fork shaped element, and the two forks are connected by the layers of VE material. When the VE materials is deformed in shear, it provides both a velocity-dependent force which provides supplemental viscous damping to the system (similar to viscous dampers), and a displacement-dependent elastic restoring force. A review of the VE material property characteristics as well as past VE damper applications are described later in this chapter.

Positioning the FCDs at these locations is efficient because they do not interfere with the architectural space of the building since they simply replace existing structural elements. A space must be provided between the slab and the FCD to allow for the damper to deform without interference. The FCD can be cast in-place during construction with an anchorage plate detail, or connected by bolting or welding to connection plates that are cast into the shear walls after the walls are poured.

The VE material layers in the FCD are engaged by the wall rotations and the relative vertical movement of the steel anchorages, which consist primarily of a shear deformation and a rotation. Figure 3.3b displays the deformed shape of two traditional RC walls coupled together with FCDs. Figures 3.3c shows a storey view of the deformed FCD and Figure 3.3d displays the exaggerated deformed shape of the FCD itself.

3.2.1 Evolution of FCD

The concept of adding dampers originated from a study by University of Toronto professors (Christopoulos, Bentz and Collins 2003) assessing the damping characteristics of a tall coupled wall tower under design in downtown Toronto. The study was conducted for structural engineering consultants, Halcrow Yolles (HY), who are a prominent design firm in Canada. The study concluded that damping characteristics in high-rise buildings are likely smaller than traditionally assumed, however concrete coupling beams can actually dissipate some energy through cracking and friction along cracks, formed in the concrete beams. However, due to this cracking the damping and stiffness decrease with the number of
CHAPTER 3: Fork Configuration Dampers (FCDs) for Use in High-Rise Buildings

Fork Configuration Dampers (FCDs) for enhanced dynamic performance of high-rise buildings

Loading cycles and the long-term (high cycle) behaviour is not well understood. Therefore damping provided by the small excursions into the nonlinear loading range by the concrete is not sufficiently reliable or large enough to provide damping for the wind response.

The study concluded with the idea for a passive damping device replacing a number of coupling beams to add damping to the overall structure, “Considering that the coupled shear wall building technique is very efficient for the design of tall buildings, further investigations on adapting this design philosophy to other higher damped elements to increase the damping characteristics of the buildings without significantly modifying the architectural/structural layout can be considered. The replacement of a number of shear link beams, with high damped elements can greatly enhance the damping characteristics”. International Patents have been filed on this new system since 2005 and are pending in a number of countries.

A research program in collaboration with structural engineers from Halcrow Yolles was established to look into the development. The topic of this thesis evolved from this research program. Displacement-activated passive dampers, such as metallic, friction or self-centering dampers, were determined not to have sufficient or reliable added damping for the frequent wind storms that occur over the service life of a building. Also, they would possibly activate in static wind loading and there could be high or low-cycle metallic fatigue issues when the dampers were activated for a large number of cycles. Pure velocity-activated...
Fork Configuration Dampers (FCDs) for Use in High-Rise Buildings

Fork Configuration Dampers (FCDs) were ruled out as well, because they would not offer resistance to static wind loading and at small amplitude vibrations would likely not provide any significant damping. Also, there was not sufficient space within the location of coupling beams to accommodate the typical size of this type of damper or geometry to properly orientate it.

Viscoelastic (VE) damping material was chosen as being the best material for frequent wind storms. Also, this material has the advantage of adding damping under all vibrations levels from ambient vibrations to severe seismic events. The configuration that was eventually chosen was the fork configuration, which activates VE material bonded vertically between the steel plates cantilevered from both walls. The steel plates are much stiffer than the VE material and the relative deformation between the walls occurs in the VE material primarily in shear. Another advantage of the configuration is that it allows for many working design configurations with the number of VE layers in parallel, the thickness of the layers and the bonded area being design variables. Figure 3.4 shows the evolution of the FCD design: both the first two concepts, Figure 3.4a with circular VE layers and Figure 3.4b with rectangular VE layers, bonded to steel plates and then anchored directly into the left and right walls. The next concept consisted of a small damper to be supplied by a damper manufacturer and site welded to a steel W-section which is then anchored into the walls. Figures 3.4d and 3.4e show the finalized designs that were developed and tested as part of this thesis.

As the design concept progressed, advice from VE material manufacturers became critical in both the damper properties and design, but also with respect to the constructibility of the device. Nippon Steel Engineering Co., NSEC, was contacted and the high damping material, ISD-111H, was selected as the best damping material for this application. The ISD-111H material has large components of stiffness and damping, can deform to large strain levels without rupture and had stable material properties over the duration of a typical wind storm. The ISD-111H material was being used at the time in seismic applications in Taiwan and the USA. Five small-scale specimens provided in-kind from NSEC with VE thicknesses of 5mm and 10mm, were tested at the University of Toronto Structural Testing Facilities (UofT) to determine the ISD-111H material properties.

Concurrently, a number of high-rise building case studies emerged from HY as potential applications of this new system. An 85-storey structure in downtown Toronto was being designed by HY, and a full design of the tower equipped with the FCDs was developed by UofT and HY. The configuration developed for the building is shown Figure 3.4d and a more detailed drawing of the FCD solution is shown in Figure 3.5. The steel plates bonded to the VE layers were installed in a slip-critical connection using high-strength bolts to built-up I-sections, which were then welded to a end-plates. Lenton weldable couplers were installed on the back of the FCD end-plates and threaded rebar is installed using a torque wrench to provide the anchorage of the damper to the concrete walls.
The design and testing of this damper is described in Chapter 4 along with a case study and overall building design of the 85-storey building equipped with these dampers described in Chapter 6. An NSERC Idea to Innovation (I2I) grant was awarded to conduct full-scale tests on the FCDs in 2008. A second damper (FCD-B) with a seismic “fuse” built into the steel elements was designed to be tested along with the original design of the FCD-A specimens. Figure 3.4e shows one FCD-B specimen and Figure 3.6 shows details of a damper design similar to the FCD-B, proposed for a planned 75-storey structure in Korea. The tests were conducted at the structures laboratory at École Polytechnique in Montreal (EPM). The test set-up was built and verified in 2009, and testing was completed in early 2010.
CHAPTER 3: Fork Configuration Dampers (FCDs) for Use in High-Rise Buildings

Figure 3.5 Proposed FCD Solution for 85-Storey High-Rise in Toronto

Figure 3.6 Proposed FCD Solution for 75-Storey High-Rise in Korea
3.3 ADDITIONAL FCD DETAILS AND DESIGNS

Through the development and incorporation of the FCD within a future project, there were many different details and designs that were developed. This section describes some of the alternative design details.

3.3.1 Slab Connection Details

A clearance is required between the FCD and the slab to allow free movement of the FCD when the building is subject to lateral loads. Figure 3.7a illustrates a traditional slab 200mm configured with the FCD. If the traditional slab is too stiff, the slab could be decreased to 135mm and still meet fireproofing requirements, as shown in Figure 3.7b. Figures 3.7c and 3.7d illustrates attempts to eliminate the stiffness effects of the slab completely. Figures 3.7c shows a slab detail with a flexible membrane that attempts to reduce the lateral stiffness of the concrete slab above the FCD interfering with the FCD stiffness. A thin precast concrete or steel panel could be introduced, as illustrated in Figure 3.7d, and it could be completely removed if inspection is required after an extreme event. For this case there may be special fire protection required. In both cases when the slab stiffness is attempted to be reduced it is assumed that the slab at all the other locations still provides sufficient rigid diaphragm action.

![Figure 3.7 Potential Slab Connection Details](image-url)
3.3.2 Wall Connection Details

Through the design process a number of additional connection details and concepts were developed. A number of the connections are shown in Figure 3.8. The full-scale connection which is proposed for the 85-storey structure that was used in the full-scale validation tests, is shown in Figure 3.8a. The proposed construction sequence using this damper for an actual building is shown in Figure 3.9.

Figure 3.8b shows an alternative connection consisting of the built-up I-section anchored directly in the concrete walls. Detailing would be similar to that required for structural steel beams coupling two walls together (Harries et al. 1993). Figure 3.8c shows a cast-in-place connection where a stiff concrete bracket is built out from the concrete wall and up to the vertical VE-steel-plate assembly. The proposed connection would require only one size of FCD for all wall clearances with concrete brackets built out and minor steel anchorage modifications to accommodate different spans. Figure 3.8d displays a welded connection where the FCD end-plates are welded to steel plates that are cast into the walls to form the FCD end-plate anchorage. The end-plate connection can be cast in before the FCD arrives on the site. The FCD is welded to the cast-in-plate with a fillet weld at the top and sides of the end-plate to form the anchorage. Figure 3.8e shows a splice-plate connection, with a variable length built-up I-section cast into the wall. The FCD is installed with bolted splice plate connections. A second bolted connection consists of a cast-in-plate with tapped holes for structural bolts forming a slip-critical connection with an end-plate with oversized holes.

The site welded and bolted connections, shown in Figures 3.8d, 3.8e and 3.8f, could be installed after the walls have been cast and possibly after the walls have undergone some differential shrinkage caused by creep, shrinkage or gravity loads. If an extreme event occurred the FCDs could be inspected for damage and if needed, could be removed and replaced.
a) Cast-In-Place End-Plate Coupler Detail

b) Built-Up I-Section Cast-In-Place Detail

c) Vertical Plates Cast-In-Place Detail
d) Variable Length Pocket Field Weld Detail

e) Splice Plate Detail

f) Bolted End-Plate Detail

Figure 3.8 Proposed Connection Details
3.3.3 Structural “Fuse” Details

A valuable feature for areas with large seismic demands would be a ductile force limiting “fuse” mechanism in series with the VE material. Through a capacity design procedure the “fuse” could be used to limit the force in the walls during an extreme event and thus prevent tearing of the VE material. With this detail it may be possible to achieve large FCD shear displacements, which would be a combination of VE material shear displacements and the “fuse” element nonlinear deformations. Figure 3.10 illustrates four such “fuse” element configurations that can be used to limit the forces transmitted to the walls. A Reduced Beam Section (RBS) “fuse” is shown in Figure 3.10a, which, if activated, would limit the bending moment introduced to the concrete walls during large shear deformations. A shear force “fuse” is shown in Figure 3.10b, which, if activated, would limit the FCD shear force once the steel section yields in shear. A slip-controlled friction “fuse” is shown in Figure 3.10c, which, if activated, would limit the shear force once the connection slips. Finally, force limiting anchor “fuses” attached to the end-plate are shown in Figure 3.10d, which, if activated, would limit the axial force in the anchorages limiting the bending moment introduced to the walls.

3.3.4 Replaceable FCD Details

Another possible beneficial detail to be incorporated in the FCD is a connection that would allow for the replaceability of the devices if an extreme event occurred. Figure 3.11 shows a FCD with a “fuse” mechanism as well as replaceable connections. Figure 3.11a illustrates a steel plate cast and anchored in the concrete walls, with tapped threaded holes for structural bolts. A slip-critical bolted connection is formed when bolts are installed to the end-plate. Figure 3.11b illustrates an FCD with oversized holes drilled in the end-plate for post-tensioning. Post-tensioning is installed using a PT connection through the PT ducts in the concrete walls and anchored at the end-plate.
Outriggers are commonly included in design of high-rise buildings to increase the lateral stiffness of the structure. The typical configuration consists of a main concrete core connected to exterior columns via relatively stiff one- or two-storey concrete walls. When subject to the lateral loads, the core is restrained against rotation by the exterior columns acting axially in tension and compression (Smith and Coull 1991 and Taranath 2010). The lateral deflections and moments in the core are decreased when the building core works in conjunction with the exterior columns. Figure 3.12 displays a core wall structure with two outriggers, one at mid-height and one at the top of the building. The FCDs are connected to the one-storey concrete wall at the exterior column, where it is expected to undergo the most deformation. In this configuration many layers of VE material with large surface area can be bonded together. The
deformations are expected to be primarily vertical, but could include horizontal and rotational VE material deformation as well. Figure 3.12c displays a figure of the deformed shape of the FCD outrigger under lateral loads. Another similar concept has been developed by the structural consulting firm, Arup. They designed a damping system that utilizes large viscous dampers configured vertically between the exterior columns and concrete wall connecting to the core wall (Smith and Wilford 2007). The system has been used in two high-rise buildings in the Philippines. Using the dampers, the concrete volume was reduced by 30% and the net floor area was increased by 2%. Viscous dampers must be inspected over time to ensure there are no leaks, cracks, fatigue or corrosion. Also, viscous dampers may not be effective under small amplitude displacements.

Figure 3.12 Potential FCD Outrigger Configuration
3.3.6 Additional FCD Structural Configurations

Additional FCD structural configurations that take advantage of the FCD acting in shear were investigated as well. These included configuring the FCD in an RC or steel eccentrically braced frame, coupling the lateral load resisting elements in moment resisting frames and braced systems configured with shear walls. Also, FCDs were investigated for steel or concrete tube systems configured in the beams of tube systems or coupling the tubes of multiple bundled tube systems.

3.4 VISCOELASTIC MATERIAL

3.4.1 Hysteretic Behaviour of VE Material

Reviews of VE material has been covered in detail in Soong and Dargush (1997), Constantinou et al. (1998) and Christopoulos and Filiatrault (2006). The following section provides a brief overview of the main features of hysteretic behaviour of VE material.

Viscoelastic materials used in civil engineering structures are typically made from acrylic copolymers or glassy elastomeric substances. They exhibit both an instantaneous elastic response and a viscous response when undergoing shear deformation that is a function of the rate of loading. For a given cycle of deformation they will return to their original shape with some energy dissipated as heat. The stress-strain relationships under harmonic oscillations are illustrated in Figures 3.13a and 3.13b.

Under harmonic oscillations it can be seen that the stress, $\tau(t)$, is proportional to that of the strain, $\gamma(t)$, but the stress leads the strain by a phase angle, $\theta$ (Mahmoodi). The symbol $G_E$ is defined as the “shear storage modulus” and is a measure of the energy stored and recovered per cycle, the symbol $G_C\omega$ is defined as the “shear loss modulus” and is a measure of the energy dissipated per cycle. Both $G_E$ and $G_C\omega$ are functions of the frequency, temperature and strain amplitude for a given hysteretic cycle. The effects of this will be discussed later.
A simple model used to represent this behaviour is the Kelvin-Voigt solid model, which is a spring and viscous damper in parallel, illustrated in Figure 3.13c. The deformation of an infinitesimal element subject to shear is illustrated in Figure 3.14a.

**a) Infinitesimal Element Subject Shear  b) Kelvin-Voigt Model  c) Force-Displacement Hysteresis**

![Figure 3.14 Infinitesimal Element Subject to Pure Shear](image)

The shear constitutive relationship for the Kelvin-Voigt solid can be expressed as (Zhang et al. 1989):

\[ \tau(t) = G_E \gamma(t) + G_C \dot{\gamma}(t) \]  

where \( \gamma(t) \) is the shear strain of the VE material at time \( t \), and \( \dot{\gamma}(t) \) is the shear strain rate at time \( t \). Eq. (3.1) is transformed into a force versus displacement relationship by multiplying Eq. (3.1) by the viscoelastic material shear thickness, \( h \), and shear area, \( A \):

\[ F(t) = ku(t) + cu(t) \]  

where \( u(t) \) is the VE shear displacement at time \( t \) and \( \dot{u}(t) \) is the VE shear displacement rate \( k \) is the elastic stiffness coefficient and \( c \) is the viscous damping coefficient and are calculated as (Soong and Dargush 1997):

\[ k = G_E A / h \]  

\[ c = G_C A / h \]  

These relations form the basis of the Kelvin-Voigt model for VE material, and is shown in Figure 3.14b. The force generated by the viscous damping element is linearly proportional to the relative velocity between the two ends of the damper material. Expressing force in terms of first principles:

\[ F(t) = A(G_E \gamma(t) + G_C \dot{\gamma}(t)) \]
The maximum VE force, \( F_0 \), for a harmonic cycle can be expressed as:

\[
F_0 = ku_0/(\cos \theta) \tag{3.6}
\]

Eq. (3.2) describes the force-displacement hysteresis shown in Figure 3.14c. Due to the viscous component, the maximum force does not occur at the maximum displacement, rather it is offset by the phase angle, \( \theta \). The area enclosed by one loop of the force-displacement hysteresis equals the energy dissipated by heat \( E_{VED} \) per cycle, which can be calculated as (Constantinou et al 1998):

\[
E_{VED} = c \pi \omega u_0^2 \tag{3.7}
\]

or:

\[
E_{VED} = \pi F_0 u_0 \sin \theta \tag{3.8}
\]

Rearranging Eq. (3.7):

\[
c = \frac{E_{VED}}{\pi \omega u_0^2} \tag{3.9}
\]

Combining Eqs. (3.8) and (3.9):

\[
\theta = \sin \left( \frac{c u_0}{F_0} \right) \tag{3.10}
\]

The elastic stiffness coefficient, \( k \), can be determined as:

\[
k = \frac{F_0}{u_0 \cos \theta} \tag{3.11}
\]

The mechanical properties of a damper can be obtained using Eqs. (3.7), (3.8) and (3.10), given experimentally measured values of \( E_{VED}, F_0 \) and \( u_0 \). From basic structural dynamics, the equivalent damping ratio of the viscoelastic damper \( \xi \) may be obtained:

\[
\xi = \frac{c}{2 \omega m} \tag{3.12}
\]

where \( \omega \) is the oscillating circular frequency of the element and \( m \) is the mass between the far ends of the element. Eq. (3.12) can be rewritten in terms of the shear storage modulus \( G_E \) and the shear loss modulus \( G_C \omega \):

\[
\xi = \frac{c_0^2}{2 \omega k} = \frac{c_0}{2k} = \frac{G_C \omega}{2G_E} \tag{3.13}
\]
The energy dissipation capacity of the viscoelastic material may also be given by the loss factor, $\eta$, defined as (Mahmoodi 1969):

$$
\eta = \frac{G_E \omega}{G_C} = 2\xi = \tan \theta
$$

(3.14)

The loss factor, $\eta$, is directly related to the lag angle between the shear stress and shear strain. Therefore, the larger the energy dissipation capacity of the VE material the larger the lag between the shear stress and the shear stress is more out of phase with the shear strain. Soong and Dargush (1997) showed that the loss factor is generally not very sensitive to the excitation frequency and is typically around unity.

Another parameter that is often defined is the complex modulus, $G^*$, which is expressed as (Soong and Dargush 1997):

$$
G^* = \frac{\tau_0}{\gamma_0} = \sqrt{(G_E^2 + (G_C^\omega)^2)}
$$

(3.15)

As mentioned before the VE material properties, $G_E$ and $G_C$, are functions of the frequency of excitation, the strain level, the ambient temperature, and the internal temperature of the VE material. These effects are shown in the results of the small-scale and full-scale tests in Chapter 4. Chapter 5 shows a method used to account for these and modelling techniques to take into account the changes in properties over the duration of a test and Chapter 6 provides design suggestions for VE property for the different loading conditions.

### 3.4.2 Past Applications

The first applications of VE dampers in civil engineering structures were to reduce wind-induced acceleration levels. Over 10,000 VE dampers were installed in each of the twin towers in the World Trade Center throughout the structure from the 10th storey through the 110th storey. They were configured between the exterior steel tube wall and the lower chords of the horizontal truss. The towers experienced a number of large wind storms and the observed performance agreed well with theoretical values. The dampers were tested every five years, up to 1996 and had been shown to age well without showing much deterioration of the VE properties (NIST 2005). During hurricane Gloria in 1978, the critical damping ratio of the building was found to be in the range of 2.5 to 3 percent of critical (Mahmoodi et al. 1987), and the added damping was shown to reduce the lateral accelerations.
CHAPTER 3: Fork Configuration Dampers (FCDs) for Use in High-Rise Buildings

A total of 260 VE dampers were installed in the Columbia SeaFirst building in Seattle to reduce the lateral vibrations (Skilling et al. 1986 and Keel and Mahmoodi 1986). The building consisted of three composite columns connected with steel braced moment frames. The VE dampers were installed along the main diagonal members in the building core. The damping ratio was predicted to increase from 0.8% to 3.2% for design level wind storms. A total of 16 large VE dampers were also installed in the Two Union Square Building in Seattle, in 1988 (Soong and Dargush 1997). The VE dampers were installed parallel to four columns in one floor.

VE dampers have only recently been used for seismic applications. In 1993, they were used for a seismic retrofit project of a 13-storey steel frame building in San Jose, CA (Crosby et al. 1994). A total of 96 VE dampers were considered the optimum retrofit solution since they provided both damping for frequent earthquakes and damping for more extreme earthquakes as well. The solution was designed to add 17% damping of critical in the first mode of vibration. A total of 64 VE dampers were added to a three storey nonductile concrete building in San Diego (Soong and Dargush 1997). They were installed in a K-brace configuration in order to reduce interstorey drifts. Two other upgrade projects that used VE dampers in California, included an 8-storey steel frame in Redwood City and a 4-storey steel frame in Los Angeles (Aiken 1997). A total of 21-VE dampers were installed in a 8-storey building with steel and concrete moment frames in Mexico City constructed in 1998 (Constantinou et al 1998). In 1991 there was also a 24-storey steel building constructed in Japan (Yokota et al. 1992) with VE dampers to enhance the seismic protection.

Figure 3.15 World Trade Center VE Application

A total of 260 VE dampers were installed in the Columbia SeaFirst building in Seattle to reduce the lateral vibrations (Skilling et al. 1986 and Keel and Mahmoodi 1986). The building consisted of three composite columns connected with steel braced moment frames. The VE dampers were installed along the main diagonal members in the building core. The damping ratio was predicted to increase from 0.8% to 3.2% for design level wind storms. A total of 16 large VE dampers were also installed in the Two Union Square Building in Seattle, in 1988 (Soong and Dargush 1997). The VE dampers were installed parallel to four columns in one floor.

VE dampers have only recently been used for seismic applications. In 1993, they were used for a seismic retrofit project of a 13-storey steel frame building in San Jose, CA (Crosby et al. 1994). A total of 96 VE dampers were considered the optimum retrofit solution since they provided both damping for frequent earthquakes and damping for more extreme earthquakes as well. The solution was designed to add 17% damping of critical in the first mode of vibration. A total of 64 VE dampers were added to a three storey nonductile concrete building in San Diego (Soong and Dargush 1997). They were installed in a K-brace configuration in order to reduce interstorey drifts. Two other upgrade projects that used VE dampers in California, included an 8-storey steel frame in Redwood City and a 4-storey steel frame in Los Angeles (Aiken 1997). A total of 21-VE dampers were installed in a 8-storey building with steel and concrete moment frames in Mexico City constructed in 1998 (Constantinou et al 1998). In 1991 there was also a 24-storey steel building constructed in Japan (Yokota et al. 1992) with VE dampers to enhance the seismic protection.
Over the last 25 years many experimental programs have been carried out on steel frames with VE dampers (Ashour and Hanson 1987, Su and Hanson 1990, Lin et al. 1991, Kirekawa et al. 1992, Aiken et al. 1993, Fujita et al. 1993, Bergman and Hanson 1993, Chang et al. 1993a and Chang et al. 1995) and reinforced concrete frames with VE dampers (Foutch et al. 1993, Lobo et al. 1993, Chang et al. 1994, Shen et al. 1995 and Lai et al. 1995). Also there have been in-situ studies of steel structures with VE dampers as well (Chang et al. 1993b and Lai et al. 1995). Most of the studies focused on seismic performance of scaled-down shear type buildings, some in actual shake table tests. The tests typically used the VE compound ISD-110 which generated results and property characterization for many different strains, frequencies and temperatures.

3.4.3 Equivalent VE Modelling Technique for FCD

The Kelvin-Voigt model described in the previous section is applicable for modelling VE dampers when the connecting element in series with the VE material is very stiff. Techniques have been developed to take into account the effects of varying brace stiffnesses in series with VE material layers (Abbas and Kelly 1993 and Kasai et al. 2006). Kasai et al. (2006) developed a procedure to account for VE damper performance when different brace stiffnesses are used, referred to as the equivalent viscoelastic model. This technique allows for the decoupling of the brace and VE material force versus displacement results when the two elements are in series. When the brace stiffness is decreased, the loss factor of the damper is decreased. The methodology used to account for the brace stiffness in series with the VE material is summarized in Figure 3.16.

A deformed shape of the damper, including the brace elastic response and the VE material response, is shown in Figure 3.16a. The damper is modelled using a linear elastic spring, representing the brace stiffness axially, and a Kelvin-Voigt model, representing the VE material layers, as illustrated in Figure 3.16b. Kasai (2006) suggests that the total damper hysteresis can be modelled as an equivalent Kelvin-Voigt model with elastic stiffness and viscous damping coefficients, $c_D$ and $k_D$, as illustrated in Figure 3.16c. The brace force versus displacement, VE hysteresis and damper hysteresis are shown in Figures 3.16d, 3.16e and 3.16f, respectively. The stiffness of the brace is represented by a linear elastic spring and expressed as $k_B$, and the elastic stiffness of the VE material is expressed as $k_{fE}$. As can be seen in the figures, the stiffness of the brace tends to flatten out the damper hysteresis; the equivalent damper hysteresis has a lower stiffness, damping and loss factor compared to the VE element alone.
As shown in these figures, the brace, VE material and damper maximum forces occur all at the same time, point 1. The maximum damper displacement occurs at point 3, while the maximum VE material displacement occurs at point 4. The equivalent damper elastic stiffness can be expressed using a combination of the individual brace and VE material elements as:

$$k_D = \frac{k_B \Gamma_{VE} k_{VE}}{k_B + \Gamma_{VE} k_{VE}}$$  \hspace{1cm} (3.16)$$

where $\Gamma_{VE}$ is a factor, which is a function of the VE material elastic stiffness, $k_{VE}$, and brace stiffness, $k_B$, and the loss factor of the VE material, $\eta_{VE}$, calculated as:

$$\Gamma_{VE} = 1 + \frac{\eta_{VE}^2}{1 + k_B/k_{VE}}$$  \hspace{1cm} (3.17)$$

The equivalent loss factor of the damper, $\eta_D$, is expressed as:

$$\eta_D = \frac{\eta_{VE}}{1 + (1 + \eta_{VE}^2) k_{VE}/k_B}$$  \hspace{1cm} (3.18)$$

The equivalent viscous damping coefficient is calculated as:

$$c_D = \frac{\eta_D k_B}{\omega_0}$$  \hspace{1cm} (3.19)$$
The modelling technique can be applied to the FCD in a shear configuration as well. The models used to represent the shear behaviour of the FCD are summarized in Figure 3.17. The deformed shape of the FCD, subject to shear displacements, $u_{FCD}$, is shown in Figure 3.17a. The spring Kelvin-Voigt model used to represent the FCD in shear assumes all shear displacement occurs at the center of the VE material. An illustration of the model is shown in Figure 3.17b.

\[ k_{FCD} = \frac{1}{k_{AL} + k_{AR}} \]  

and each assembly stiffness may be determined approximately using beam theory approximations for all the steel elements cantilevered from the wall. Since the steel elements likely do not follow beam theory it may be more reasonable to conduct an FE analysis with solid elements using software such as ANSYS. However, the best method to determine the assembly stiffness would be to measure it directly from experiments. This technique was used to obtain the stiffness values from the full-scale FCD tests.
The $k_{VE}$, $c_{VE}$ and $\eta_{VE}$ parameter values are calculated using the manufacturer material properties and the equivalent FCD properties, $k_{FCD}$, $c_{FCD}$ and $\eta_{FCD}$, in shear, at the center of the clearspan. They are calculated using the same formulae as the brace configuration as:

$$k_{FCD} = \frac{k_A \Gamma_{VE} k_{VE}}{k_A + \Gamma_{VE} k_{VE}}$$  \hspace{1cm} (3.21)

$$\Gamma_{VE} = 1 + \frac{\eta_{VE}^2}{1 + k_A/k_{VE}}$$  \hspace{1cm} (3.22)

$$\eta_{FCD} = \frac{\eta_{VE}}{1 + (1 + \eta_{VE})k_{VE}/k_A}$$  \hspace{1cm} (3.23)

$$c_{FCD} = \frac{\eta_{FCD} k_{FCD}}{\omega_0}$$  \hspace{1cm} (3.24)

These formulae can be used to model the FCD behaviour in high-rise buildings using commercially available software.

### 3.4.4 Viscoelastic-Plastic Modelling Techniques for FCD

A beneficial feature of the FCD is the “fuse” mechanism which could be activated in a large seismic event. The “fuse” is capacity designed to limit forces in the walls and to prevent damage or failure of the VE material. A summary of the technique used for modelling the viscoelastic-plastic behaviour is illustrated in Figure 3.18. Figure 3.18a illustrates the deformed shape of the FCD after fuse activation. In this case the fuse is an RBS connection yielding in flexure. Figure 3.18b illustrates the Kelvin-Voigt model in series with the elasto-plastic model. Again, the VE material is lumped at a rigid offset at the center of the FCD clearspan. To demonstrate the nonlinear behaviour, a harmonic displacement signal for 4 full cycles with the same frequency is applied to the FCD. The FCD displacement amplitude is increased at every cycle; the FCD displacement amplitude is 0.25$u_{\text{Max}}$, 0.5$u_{\text{Max}}$, 0.75$u_{\text{Max}}$ and $u_{\text{Max}}$, as shown in Figure 3.18c. Figures 3.18d, 3.18e and 3.18f show the assembly “fuse” hysteresis, VE material hysteresis and the total FCD hysteresis, respectively. In the first cycle the flexural “fuse” does not yield and the deformation is mainly in the VE material. The behaviour could be modelled using the equivalent FCD model described in the previous section. During the second cycle the RBS “fuse” is activated and when the max force, $F_{\text{Max}} = M_{\text{Max}}/(2L_{\text{RBS}})$, is reached the VE material displacement is clamped.

For a brace configuration with a flat yield plateau, $F_y$, the maximum VE displacement can be calculated as (Kasai 2002):

$$u_{VEMax} = F_y/k_{VE}(f)$$  \hspace{1cm} (3.25)
For the case with a less pronounced yield plateau, such as a RBS connection, the maximum displacement is slightly more. This is because after yield, the stiffness is not zero and therefore there is an interaction between the Kelvin-Voigt model and the elasto-plastic model. It can be seen that the FCD could achieve large displacement amplitudes and dissipate large amounts of energy. The “fuse” concept as described above could potentially be a very beneficial detail for the FCD. It could allow the FCD to be designed to add inherent damping for small amplitude vibrations such as wind loads and provide a “fuse” mechanism under much larger loads.

**Figure 3.18 FCD Viscoelastic-Plastic Model in Shear**

**3.4.5 Optimization of FCD Locations**

When VE FCDs are installed in lieu of coupling beams throughout the height of the structure they add distributed linear viscous damping. The total energy dissipated by $N$ FCDs for the $i^{th}$ mode and $j^{th}$ locations, is expressed as:

$$E_{VEDi} = \sum_{j=1}^{N} c(f_i)\pi\omega_i u_{0ij}^2$$  \hspace{1cm} (3.26)
The equivalent viscous damping can be expressed as

\[ \zeta_{eqi} = \frac{1}{4\pi} \frac{E_{VFDi}}{E_{Si}} \]  \hspace{1cm} (3.27) 

where \( E_{Si} \) is the strain energy at the maximum displacement of the \( i^{th} \) mode of vibration and is expressed as (Chang et al. 1991 and Constantinou and Symans 1992):

\[ E_{Si} = \frac{1}{2} \{ \phi_i \}^T [K] \{ \phi_i \} \]  \hspace{1cm} (3.28) 

where \( \{ \phi_i \} \) is the \( i^{th} \) mode of vibration and \([K]\) is the stiffness matrix. The optimal locations of the FCDs for a given mode of vibration is where the maximum amount of energy is dissipated for that mode of vibration. This occurs when the displacement of the FCDs are maximized. For the first mode of vibration, the maximum shear displacements occur, roughly, in the same profile as the shear force described by the continuum approach in the previous chapter.

For coupled wall building projects designed currently, the lateral load resisting schemes are extremely irregular and are typically modelled using 3-D FE modelling software. For this reason implementation in 2-dimensional portal frame models can give important trends, however, may not be useful for the actual design. Also, damping in torsional modes may be the governing design requirement, and when this is the case, 3-D FE models are necessary. For this reason a practical means for evaluating the supplemental damping and optimization of damper locations is to use a trial and error method placing FCDs in locations that undergo the most shear deformations for a given mode shape. An iterative procedure is used to evaluate the effects of the added damping until an optimum solution is obtained.

### 3.5 Conclusion

The FCD concept was introduced in this chapter. The benefits include added distributed viscous damping, which reduces the dynamic response to frequent wind storms or small earthquakes. For high-seismic regions a “fuse” is introduced, and capacity designed to ensure damage or failure of the VE material does not occur, while limiting the forces introduced into the concrete walls. Further to this, a replaceable connection was introduced that could be replaced if, after a large seismic event, it was determined that the “fuse” element had yielded. There were many details described including proposed slab details for the FCD and wall anchorage connection details. Details for the proposed “fuse” elements and replaceable connection were illustrated as well. The outrigger FCD concept was also introduced as a potential configuration of the FCDs. VE material property models were introduced and past applications of VE dampers were discussed as well.
4.1 INTRODUCTION

In this chapter, the results from two different test programs are presented. The first set of tests was conducted on small-scale VE damper specimens at the University of Toronto Structural Testing Facilities (UofT), which were used to validate the VE material properties under realistic loading conditions of high-rise buildings. Next a second set of tests was conducted on two different full-scale FCD samples at the École Polytechnique in Montreal (EPM). The full-scale FCD tests were used to validate the overall system performance based on the kinematic behavior of coupled wall systems, wall anchorage and VE material behavior during actual design level loadings. The FCD samples were designed for the application in two high-rise building projects in Toronto and Vancouver; both samples were designed using a capacity design procedure to ensure a ductile and predictable performance at large load levels. The tests include harmonic tests which characterized the FCD and VE material properties under a range of frequencies and strains. Also, analytically determined FCD response time-histories were conducted for wind and earthquake vibrations. The FCDs were then cycled to failure either dynamically or statically, establishing the ultimate and hysteretic behaviour of the FCDs connection. Lastly, recommendations for modelling and design of FCDs are made based on conclusions from the tests.

4.2 SMALL-SCALE DAMPER AND VISCOELASTIC MATERIAL CHARACTERIZATION AT UNIVERSITY OF TORONTO (UOFT)

4.2.1 Test Configuration and Damper Design

Five small-scale viscoelastic damper (SSVD) specimens were provided by Nippon Steel Engineering Co. (NSEC) (Figure 4.1) to validate the properties of the VE material used for the full-scale damper specimens. The SSVDs consisted of four layers of 150x150 mm VE ISD-111H material bonded in between steel plates. Three of the specimens had 5 mm thick VE material layers and two of the specimens had 10 mm thick layers VE material layers.
The 3 - 5 mm thick VE material specimens that were tested were labelled UT-VEM5-1, UT-VEM5-2, UT-VEM5-3 and the 2 - 10 mm VE material specimens that were tested were labelled as UT-VEM10-1, UT-VEM10-2. Drawings of the two SSVDs are shown in Figures 4.1 and 4.2.
The SSVDs were tested by applying axial loading along their longitudinal axis of the SSVDs in an MTS universal testing machine (UTS), capable of applying both monotonic or cyclical loading. The capacity of the testing machine is 245kN in both tension and compression, with a maximum stroke of ±80mm and a maximum velocity of approximately 40mm/s. Axial force measurements were obtained from the internal load cell and displacement measurements were obtained using an internal potentiometer and two external LVDTs mounted to rigid steel angles bolted to the damper end-plates. A handheld contact thermometer with a type K thermocouple was used to measure the temperature of the VE material at the exposed surfaces before and after tests. Unfortunately, the handheld thermometer relied on measurements of the VE material temperature only at the exposed surface of the VE material. Photos of the complete test setup along with the UT-VEM10-1 specimen deformed at 200% shear strain are shown in Figure 4.3.

4.2.2 Loading Protocol

The loading protocol was developed to validate the material properties of the VE ISD-111H in the frequency range of interest and the projected strain levels for which the FCD would be designed for. Unfortunately, the test control system did not permit displacement controlled time-history signals, however, displacement controlled harmonic cyclic tests and quasi-static tests were possible. The entire list of tests for all specimens as well as the test results are located in Appendix C. Table 4-1 illustrates the loading protocols for each of the dampers. The test descriptions of the harmonic material characterization, wind loading, earthquake loading and control testing are outlined in the following Section 4.2.2.1.
4.2.2.1 Material Characterization Test Protocols

A set of material characterization tests were used to obtain the mechanical properties of the VE material under realistic loading conditions for FCDs that might be installed in a real high-rise building. The test protocol is described below.

The goal of the first test was to measure the response of the damper material when subjected to a range of frequencies and strains under harmonic loading. Table 4-2 displays the tested protocol for samples VEM5-3 and VEM10-1.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Test Set #1</th>
<th>Test Set #2</th>
<th>Test Set #3</th>
</tr>
</thead>
<tbody>
<tr>
<td>UT-VEM5-1</td>
<td>Material Characterization</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>UT-VEM5-2</td>
<td>Wind Loading</td>
<td>Earthquake Loading</td>
<td>Control Tests</td>
</tr>
<tr>
<td>UT-VEM5-3</td>
<td>Material Characterization</td>
<td>Control Tests</td>
<td>-</td>
</tr>
<tr>
<td>UT-VEM10-1</td>
<td>Material Characterization</td>
<td>Control Tests</td>
<td>-</td>
</tr>
<tr>
<td>UT-VEM10-2</td>
<td>Wind Loading</td>
<td>Earthquake Loading</td>
<td>Control Tests</td>
</tr>
</tbody>
</table>

4.2.2.2 Wind Testing Protocol

4.2.2.2.1 Wind Characteristics

The two governing wind load cases, along- and across-wind were described in Section 2.4. The total along-wind building response is a combination of static and dynamic components, whereas the across-wind response is only dynamic. The torsional response is also a concern in high-rise building design, however the torsional response characteristics are predominantly dynamic and the response of individual members is similar to the across-wind response. The peak structural displacement response, $R_{Max}$, for the along and
across-wind design cases may be expressed as:

\[ R_{\text{MaxAlong}} = \bar{R} \pm g_f \sigma_R \]  
\[ R_{\text{MaxAcross}} = \pm g_f \sigma_R \]  

where \( \bar{R} \) is a quasi-static mean component for the duration of the storm and \( g_f \sigma_R \) is the dynamic component, \( g_f \) is a gust factor which typically ranges from 3.5 to 4.0, and \( \sigma_R \) is the root mean square response. The ULS response is approximately 1.35 times greater than the SLS response (Simiu and Scanlan, 1996). The maximum damper strain response, \( \gamma_{\text{Max}} \), for an hour of loading (an hour being the duration of the typical design basis wind storm) may be expressed in the same manner as:

\[ \gamma_{\text{MaxAlong}} = \bar{\gamma} \pm g_f \sigma_{\gamma_{\text{Along}}} \]  
\[ \gamma_{\text{MaxAcross}} = \pm g_f \sigma_{\gamma_{\text{Across}}} \]  

NSEC suggested a design service limit of \( \gamma_{\text{Max}} = \pm 100\% \) for the 1 in 10 year SLS storm. Figure 4.4 displays the damper strain time history of an FCD in a coupled wall tall building with a natural period of vibration of \( T_n = 5.24 \) seconds subjected to the SLS across-wind vibration for one hour. The maximum strain is \( \gamma_{\text{Max}} = \pm 100\% \) and the root mean square strain is \( \sigma_{\gamma} = \pm 28.5\% \), using an assumed gust factor of \( g_f = 3.5 \). The damper strain is above 25%, for 36.5% of the time, above 50%, for 8.7% of the time and above 75%, for 1.1% of the time. The number of times the damper strain is greater than 75% is approximately 40 times which would represent 20 full cycles or an equivalent duration of approximately 100 seconds. The mean response amplitude, \( \bar{R} \), in the along-wind direction of loading is assumed to be 60% of the total loading, but is very hard to predict without testing the structure in a wind-tunnel.

Figure 4.4 FCD SLS Wind Storm Response
4.2.2.3 Proposed Wind Testing Protocol

For a damper subjected to a displacement controlled sinusoidal input, the maximum strain, \( \gamma_0 \), can be expressed as a function of the root mean square strain, \( \sigma_\gamma \), as:

\[
\gamma_0 = \sqrt{2} \sigma_\gamma
\]  
(4.5)

The maximum dynamic strain, \( \gamma_{Max} \), is expressed as:

\[
\gamma_{Max} = g_f \sigma_\gamma
\]  
(4.6)

Since the energy dissipation is a function of the root mean square strain, \( \sigma_\gamma \), using Eq. (4.5) and solving for \( \sigma_\gamma \) using a gust factor of \( g_f = 3.7 \) and substituting into Eq. (4.6) leads to the harmonic sine wave strain amplitude, \( \gamma_0 \):

\[
\gamma_0 = \frac{\sqrt{2}}{3.7} \gamma_{Max}
\]  
(4.7)

Table 4-3 and Figure 4.5 display the complete wind loading protocol tested at frequencies of \( f_0 = 0.1, 0.15 \), and \( 0.2 \) Hz. The wind loading protocol was defined to capture both the amount of the energy dissipated by the FCDs and the peak strain response of the FCD during wind loading. As such, a long duration component representative of the RMS response was included in the protocol to represent the total amount of energy dissipated as well as a high-amplitude component representing large strain levels. There is also a pre- and post-storm component of low amplitude to characterize the VE material at low amplitudes as the wind storm builds strength. Prior to applying the dynamic loading protocol that is illustrated in Figure 4.5c and d, the static deformation is applied over 15 minutes to ensure that no dynamic effects are introduced in the system as a result of the application of the static deformation. The complete results from the wind testing protocol are included in Appendix C.
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

TABLE 4-3 Wind Loading Protocol

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Pre Storm</th>
<th>RMS Component</th>
<th>Max Component</th>
<th>RMS Component</th>
<th>Post Storm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Time (min)</td>
<td>15</td>
<td>27</td>
<td>6</td>
<td>27</td>
</tr>
<tr>
<td>Across SLS</td>
<td>γ₀(%)</td>
<td>±20</td>
<td>±40</td>
<td>±100</td>
<td>±40</td>
</tr>
<tr>
<td>Along SLS</td>
<td>γ₀(%)</td>
<td>75±10</td>
<td>75±20</td>
<td>75±50</td>
<td>75±20</td>
</tr>
<tr>
<td>Across ULS</td>
<td>γ₀(%)</td>
<td>±25</td>
<td>±50</td>
<td>±135</td>
<td>±50</td>
</tr>
<tr>
<td>Along ULS</td>
<td>γ₀(%)</td>
<td>100±12.5</td>
<td>100±25</td>
<td>100±70</td>
<td>100±25</td>
</tr>
</tbody>
</table>

Figure 4.5 Wind Loading Protocol
4.2.2.4 Earthquake Loading Protocol

Based on preliminary studies of prototype buildings with FCDs, the maximum strain predicted during design level earthquakes (DLEs) and maximum credible earthquakes (MCEs) was $\gamma_{\text{Max}} = 250\%$ and $\gamma_{\text{Max}} = 400\%$, respectively. Although larger strains were tested, NSEC suggested that the strains that the damper would be subjected to be limited to a maximum strain of approximately $\gamma_{\text{Max}} = 400\%$. In a rare earthquake, the FCDs could reach large strain levels if all connections were to remain linear elastic, however only one or two such large excursions would be expected, with the remainder of the response characterized by the large strains and the remainder being smaller cycles or free vibration of the structure in the first few modes of vibration. To represent this, the dampers were tested harmonically at the expected building natural frequency and at a maximum strain of $\gamma_0 = 400\%$ for a small number of cycles. These tests were expected to represent a very conservative upper bound of strain and number of cycles for FCDs during a large earthquake. If larger strains are expected during earthquake vibrations it would be beneficial to incorporate a mechanism that limits the VE material displacements.

4.2.2.5 Control Test Protocol

At the end of each test a quasi-static ramp loading up to a strain of $\gamma_0 = \pm 100\%$ for the VEM10-2 sample, and $\gamma_0 = \pm 200\%$ for the VEM5-2, sample were conducted to determine if there was a loss of stiffness or any damage in the VE material had occurred during a test.

4.2.3 Small-Scale Test Results

The procedure used for determining the mechanical properties of the Kelvin-Voigt VE solid model material properties from the test data are outlined in the steps below. The Kelvin-Voigt VE model has been described in Section 3.4.

**STEP 1)** An individual hysteretic cycle is defined when the VE displacement $u$, crosses zero. The energy dissipated per cycle, $E_{\text{VED}}$, is determined by integrating the force obtained from the MTS internal load cell, $F$, over the VE displacement, $u$, for that cycle:

$$E_{\text{VED}} = \int F du$$

**STEP 2)** The viscous damping coefficient of the VE material, $c$, is determined using the average displacement amplitude for each cycle, $u_0$, and the circular frequency $\omega_0 = 2\pi f_0$, respectively:

$$c = \frac{E_{\text{VED}}}{\pi \omega_0 u_0^2}$$

**STEP 3)** The phase angle, $\theta$, is obtained by using the harmonic offset of the average force amplitude, $F_0$, and average shear displacement amplitude, $u_0$:

$$\theta = \arcsin\left(\frac{c \omega_0 u_0}{F_0}\right)$$
STEP 4) The elastic stiffness coefficient, \( k \), is then calculated as:

\[
k = \frac{F_0}{u_0 \cos \theta}
\]  

(4.11)

STEP 5) From the elastic stiffness coefficient, \( k \), the shear storage modulus of the VE material, \( G_E \), is obtained by multiplying the elastic stiffness coefficient by the height of the VE layers, \( h \), divided by the total area of the VE layer undergoing shear, \( A \):

\[
G_E = \frac{k h}{A}
\]  

(4.12)

STEP 6) The shear loss modulus, \( G_C \omega \), is calculated by multiplying the viscous damping coefficient of the VE material, \( c \), by the height of the VE layers, \( h \), divided by the total area of the VE layer undergoing shear, \( A \), multiplied by the circular frequency \( \omega_0 = 2 \pi f_0 \):

\[
G_C \omega = \frac{c h}{A} \omega_0
\]  

(4.13)

STEP 7) The loss factor of the VE material, \( \eta \), is calculated as:

\[
\eta = \frac{G_C \omega}{G_E}
\]  

(4.14)

4.2.3.1 Results from Material Characterization

The Kelvin-Voigt material properties that were measured during the first cycle of vibration for the test samples VEM5-3 and VEM10-1 are given in Tables 4-4 and 4-5, respectively. The VE surface temperature measured right before the first cycle of vibration is the most accurate temperature measurement, because the VE material was at the ambient room temperature.

| TABLE 4-4 VEM5-3: Material Characterization: Results From First Cycle |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| \( f_0 \) | \( T_{VE0} \) | \( G_E \) | \( G_C \omega \) | \( \eta \) | \( f_0 \) | \( T_{VE0} \) | \( G_E \) | \( G_C \omega \) | \( \eta \) | \( f_0 \) | \( T_{VE0} \) | \( G_E \) | \( G_C \omega \) | \( \eta \) |
| Hz | °C | MPa | MPa | | °C | MPa | MPa | | °C | MPa | MPa | |
| 0.1 | 20.3 | 0.103 | 0.083 | 0.791 | 22.7 | 0.093 | 0.064 | 0.691 | 21.2 | 0.094 | 0.076 | 0.801 | 23.5 | 0.081 | 0.062 | 0.767 |
| 0.15 | 20.0 | 0.137 | 0.120 | 0.875 | 25.1 | 0.093 | 0.069 | 0.747 | 21.7 | 0.111 | 0.097 | 0.871 | 21.7 | 0.105 | 0.091 | 0.862 |
| 0.2 | 20.7 | 0.134 | 0.122 | 0.905 | 25.0 | 0.101 | 0.079 | 0.786 | 21.4 | 0.128 | 0.116 | 0.907 | 22.8 | 0.111 | 0.096 | 0.868 |
| 0.25 | 19.4 | 0.170 | 0.167 | 0.986 | 21.5 | 0.144 | 0.131 | 0.912 | 23.2 | 0.129 | 0.117 | 0.906 | 22.3 | 0.125 | 0.113 | 0.905 |
| 0.3 | 20.3 | 0.161 | 0.155 | 0.962 | 21.0 | 0.164 | 0.157 | 0.957 | 22.1 | 0.146 | 0.136 | 0.927 | 20.6 | 0.153 | 0.143 | 0.937 |
4.2.3.1.1 Frequency Dependence

The mechanical properties of the VE material are dependent on the frequency at which the material is loaded. Figures 4.6a and 4.6b show the first cycle of samples UT-VEM5-3 and UT-VEM10-1, respectively, to a constant strain amplitude of \( \gamma_0 = 50\% \), with varying frequencies of \( f_0 = 0.1, 0.2 \) and \( 0.3\, \text{Hz} \), along with the VE material temperature at the beginning of each test.

![Figure 4.6 Frequency Dependency of VE Material Response at a Strain of 50% for the First Cycle of Loading](image)

The hysteresis for both samples show an increase in both the elastic stiffness and the damping when the frequency of excitation, \( f_0 \), is increased. Figures 4.7a and 4.7b display the elastic shear modulus \( G_E \) and the loss factor \( \eta \), respectively, of the first test cycle for strain amplitudes of \( \gamma_0 = 50, 100, 150 \) and \( 200\% \) and the three frequencies \( f_0 = 0.1, 0.2 \) and \( 0.3\, \text{Hz} \) which shows an increase in both material properties, \( G_E \) and \( \eta \), for all strains as the excitation frequency increases.

<table>
<thead>
<tr>
<th>( f_0 ) (Hz)</th>
<th>( T_{VE0} ) (°C)</th>
<th>( G_E ) (MPa)</th>
<th>( G_C ) (MPa)</th>
<th>( \eta ) (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>24.6</td>
<td>0.086</td>
<td>0.059</td>
<td>0.684</td>
</tr>
<tr>
<td>0.15</td>
<td>24.0</td>
<td>0.107</td>
<td>0.081</td>
<td>0.758</td>
</tr>
<tr>
<td>0.2</td>
<td>25.4</td>
<td>0.107</td>
<td>0.082</td>
<td>0.765</td>
</tr>
<tr>
<td>0.25</td>
<td>21.6</td>
<td>0.153</td>
<td>0.138</td>
<td>0.903</td>
</tr>
<tr>
<td>0.3</td>
<td>20.8</td>
<td>0.165</td>
<td>0.149</td>
<td>0.905</td>
</tr>
<tr>
<td>0.4</td>
<td>20.8</td>
<td>0.184</td>
<td>0.167</td>
<td>0.932</td>
</tr>
<tr>
<td>0.5</td>
<td>20.0</td>
<td>0.202</td>
<td>0.185</td>
<td>0.960</td>
</tr>
<tr>
<td>0.6</td>
<td>19.2</td>
<td>0.220</td>
<td>0.203</td>
<td>0.988</td>
</tr>
<tr>
<td>0.7</td>
<td>18.4</td>
<td>0.238</td>
<td>0.221</td>
<td>1.016</td>
</tr>
<tr>
<td>0.8</td>
<td>17.6</td>
<td>0.256</td>
<td>0.239</td>
<td>1.044</td>
</tr>
<tr>
<td>0.9</td>
<td>16.8</td>
<td>0.274</td>
<td>0.257</td>
<td>1.072</td>
</tr>
<tr>
<td>1.0</td>
<td>16.0</td>
<td>0.292</td>
<td>0.275</td>
<td>1.099</td>
</tr>
</tbody>
</table>

**TABLE 4-5 VEM10-1: Material Characterization: Results From First Cycle**

The mechanical properties of the VE material are dependent on the frequency at which the material is loaded. Figures 4.6a and 4.6b show the first cycle of samples UT-VEM5-3 and UT-VEM10-1, respectively, to a constant strain amplitude of \( \gamma_0 = 50\% \), with varying frequencies of \( f_0 = 0.1, 0.2 \) and \( 0.3\, \text{Hz} \), along with the VE material temperature at the beginning of each test.

The hysteresis for both samples show an increase in both the elastic stiffness and the damping when the frequency of excitation, \( f_0 \), is increased. Figures 4.7a and 4.7b display the elastic shear modulus \( G_E \) and the loss factor \( \eta \), respectively, of the first test cycle for strain amplitudes of \( \gamma_0 = 50, 100, 150 \) and \( 200\% \) and the three frequencies \( f_0 = 0.1, 0.2 \) and \( 0.3\, \text{Hz} \) which shows an increase in both material properties, \( G_E \) and \( \eta \), for all strains as the excitation frequency increases.
4.2.3.1.2 Strain Dependence

As the strain amplitude increases $G_E$ decreases and $\eta$ remains relatively constant for constant frequency and temperature. Figures 4.8a and 4.8b display the first response cycle hysteresis of samples UT-VEM 5-3 and UT-VEM 10-1, respectively, at a constant frequency of $f_0 = 0.2\text{Hz}$ and at four different strain amplitudes, $\gamma_0 = 50, 100, 150, 200\%$, respectively.

Figure 4.8 Strain Dependency of VE Material Response at a Frequency of 0.2 Hz for the First Cycle of Loading
Both hystereses show a slight decrease in the elastic stiffness and damping as the strain amplitudes increase. Figures 4.9a and 4.9b display the elastic shear modulus $G_E$ and the loss factor $\eta$, respectively, for the first test cycle at three frequencies $f_0 = 0.1, 0.2$ and $0.3\text{Hz}$ for strain levels of $\gamma_0 = 50, 100, 150$ and $200\%$ which shows a decrease in $G_E$ while $\eta$ remains relatively constant for all frequencies as the strain amplitude increases.

![Figure 4.9 Strain Dependence of the Average Properties from the First Cycle of the Material Characterization Tests](image)

**4.2.3.1.3 Ambient Temperature Dependence**

At a constant strain amplitude and frequency, as the ambient temperature increases the $G_E$ and $\eta$ properties decrease. Figures 4.10a and 4.10b show the first cycle response at a strain amplitude of $\gamma_0 = 50\%$ at frequencies of $f_0 = 0.1$ and $0.2\text{Hz}$ respectively.

There is a decrease in the elastic stiffness and damping when the ambient temperature increases. Unfortunately the effect of the ambient temperature could not be more thoroughly studied in this phase of the testing program, because the room temperature could not be controlled and therefore the range of ambient temperature was only from about $T = 20^\circ\text{C}$ to $T = 25^\circ\text{C}$. Also, there were multiple tests a day and often the internal VE material temperature was not able to reach the ambient temperature throughout the thickness before the next test, so the temperature gradient was unknown.
4.2.3.1.4 Dependence on Internal Temperature

As VE material is subjected to vibrations, it undergoes shear deformation and dissipates the vibrational energy as heat, increasing the internal temperature of the VE material from the ambient temperature, $T_{amb}$. Chapter 5 explains how the internal temperature can be incorporated into the analytical models to capture the change in material properties over the duration of a load.

Figures 4.11a and 4.11b show the force-displacement response of tests at a frequency of $f_0 = 0.1\,Hz$ and strain amplitudes of $\gamma_0 = 50\%$ and $200\%$, respectively while Figures 4.11c, 4.11d show test results for a frequency of $f_0 = 0.3\,Hz$ and strain amplitudes of $\gamma_0 = 50\%$ and $200\%$, respectively. As can be seen in the figures the elastic stiffness and damping decrease over the duration of the tests.

Figures 4.12a and 4.12b shows the shear storage modulus, $G_E$, and loss factor, $\eta$, over the duration of the test. At the beginning of the test, the VE material temperature increases more in each cycle of vibration, which is related to the energy dissipation during each cycle, approaching approximately a steady-state response towards the end of the tests, which is controlled largely by the heat convection and conduction of the steel and VE layers. Also, it can be seen that tests at a larger strain amplitude of $\gamma_0 = 200\%$ dissipate more energy and approach a steady state temperature more quickly than the tests conducted at a the smaller strain amplitude of $\gamma_0 = 50\%$ for both frequencies.

Note the ambient steel surface and VE material surface temperature at the start and the end of each test were measured using the handheld thermometer are reported in Table 4-6; note also that the VE material temperature measured at the end of the two-hour test reached $T_{VE} = 45.6^\circ C$ for a frequency $f_0 = 0.3\,Hz$ at a strain of $\gamma_0 = 200\%$. 

Figure 4.10 Ambient Temperature Dependency of VE Material Response at a Strain of 50% for the First Cycle of Loading
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

TABLE 4-6  UT-VEM10-1: Material Characterization: Handheld Thermometer Temperature Measurement at Beginning and End of Tests

<table>
<thead>
<tr>
<th>$f_0$</th>
<th>$\gamma_0 = 50%$</th>
<th>$\gamma_0 = 200%$</th>
<th>$\gamma_0 = 50%$</th>
<th>$\gamma_0 = 200%$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Hz$</td>
<td>$t = 0s$ $T_{Ambient}$</td>
<td>$T_{Steel}$</td>
<td>$T_{VE}$</td>
<td>$t = 7200s$ $T_{Ambient}$</td>
</tr>
<tr>
<td>0.1</td>
<td>24.6</td>
<td>24.6</td>
<td>24.6</td>
<td>24.7</td>
</tr>
<tr>
<td>0.3</td>
<td>20.8</td>
<td>20.8</td>
<td>20.8</td>
<td>21.3</td>
</tr>
</tbody>
</table>

Figure 4.11 Material Characterization Hysteresis: UT-VEM10-1
4.2.3.2 Results from Representative Wind Loading Test Protocols

4.2.3.2.1 Across-wind

Figures 4.13a and 4.13b show the across-wind SLS hystereses at frequencies $f_0 = 0.1 \text{Hz}$ and $0.2 \text{Hz}$, respectively and Figures 4.13c and 4.13d show the across-wind ULS hysteresis at frequencies $f_0 = 0.1 \text{Hz}$ and $0.2 \text{Hz}$, respectively. The maximum strain component, as shown in Figures 4.5a and 4.5b, shows the maximum reduction in elastic stiffness and damping, while the pre- and post-storm cycles show a much smaller reduction in elastic stiffness and damping.

Figures 4.14a and 4.14b show the elastic shear modulus and loss factor versus time for the across-wind SLS and ULS tests. During the maximum cycle subset, the $G_E$ and $\eta$ properties are the lowest, because the strain level is the largest and the energy dissipated during these cycles are the highest. Also at this subset the temperature increase in the VE material and corresponding decrease in $G_E$ and $\eta$ are the largest. The pre-storm $G_E$ and $\eta$ properties are the largest, followed by the values measured during the first RMS set, each of which have an increase in temperature in the VE material which in turn decreases the $G_E$ and $\eta$ properties. In the second RMS set and the post-storm set the $G_E$ and $\eta$ properties increase slightly, likely caused by a decrease in temperature of the VE material caused by conduction or convection between the VE material and the steel plates while the VE material dissipates less energy at these lower amplitude cycles.
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

113

Fork Configuration Dampers (FCDs) for Enhanced Dynamic Performance of High-Rise Buildings

Figure 4.13 Across-wind Hysteresis: UT-VEM5-2

- a) Across-wind SLS, $f_0 = 0.1\text{Hz}$
  - $F_{w_0} = 8.99\text{kN}$
  - $T_e = 24.7^\circ\text{C}$

- b) Across-wind SLS, $f_0 = 0.2\text{Hz}$
  - $F_{w_0} = 11.40\text{kN}$
  - $T_e = 24.7^\circ\text{C}$

- c) Across-wind ULS, $f_0 = 0.1\text{Hz}$
  - $F_{w_0} = 12.36\text{kN}$
  - $T_e = 22.7^\circ\text{C}$

- d) Across-wind ULS, $f_0 = 0.2\text{Hz}$
  - $F_{w_0} = 15.55\text{kN}$
  - $T_e = 23.5^\circ\text{C}$

Figure 4.14 Across-wind VE Material Properties: UT-VEM5-2

- a) Shear Storage Modulus
- b) Loss Factor

Fork Configuration Dampers (FCDs) for Enhanced Dynamic Performance of High-Rise Buildings
4.2.3.2.2 Along-wind Tests

The UTS displacement amplitude was manually controlled to the mean quasi-static strain $\gamma_{Static} = 75\%$ for the SLS test and $\gamma_{Static} = 100\%$ for the ULS test and held for approximately 15 minutes, and then harmonic cycles were applied about this mean static displacement. Figures 4.15a and 4.15b show the along-wind SLS hysteresis at frequencies $f_0 = 0.1\text{Hz}$ and $0.2\text{Hz}$, respectively, and Figures 4.15c and 4.15d show the along-wind ULS hysteresis at frequencies $f_0 = 0.1\text{Hz}$ and $0.2\text{Hz}$, respectively.

Figures 4.12b and 4.12c display the dynamic material properties of the VE material, which are similar to the properties obtained during the dynamic tests without the static components. The along-wind force can be expressed using the method of superposition as:

$$F_{Along}(t) = k_{Static}u_{Static} + ku_{Dynamic}(t) + cu_{Dynamic}(t)$$  \hspace{1cm} (4.15)

where $k_{Static}$ and $u_{Static}$ are the mean elastic stiffness coefficient and the mean displacement, respectively, and $k$, $c$, $u_{Dynamic}$ and $u_{Dynamic}$ are the dynamic elastic shear stiffness, viscous damping coefficient, dynamic shear displacement and the dynamic velocity, respectively.

Figure 4.16a displays the mean static shear stiffness with time. The mean stiffness recorded during the test shows that the mean static stiffness is not as high as the dynamic material stiffness and reduces with time. If there is a significant static component in the wind load, for a given mean static force, the mean static displacement may be large and may actually increase through the duration of a storm. This has design implications for loads on the adjacent structural elements and the drift limits which can be governed by loads in the along-wind direction, and will be discussed in Chapter 6.
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

Figure 4.15 Along-wind Hysteresis: UT-VEM5-2

a) Along-wind SLS, $f_s = 0.1\text{Hz}$

b) Along-wind SLS, $f_s = 0.2\text{Hz}$

Figure 4.16 Along-wind VE Material Properties: UT-VEM5-2

c) Along-wind ULS, $f_s = 0.1\text{Hz}$

d) Along-wind ULS, $f_s = 0.2\text{Hz}$

Figure 4.16 Along-wind VE Material Properties: UT-VEM5-2

a) Mean Stiffness

b) Shear Storage Modulus

c) Loss Factor
4.2.3.3 Earthquake Tests

The response of the UT-VEM5-2 specimen cycled to a strain amplitude of $\gamma_0 = 400\%$ at a frequency of $f_0 = 0.2Hz$ for 6 cycles is displayed in Figure 4.17a.

![Figure 4.17 MCE VE Material Properties: UT-VEM5-2](image-url)

As shown in Figure 4.17a, there is a large drop in stiffness and damping for each cycle of vibration with a large decrease in the shear modulus, caused by the temperature increase that takes place in each cycle while the loss factor is approximately constant.

4.2.3.4 Control Tests

In-between each test, a control test was conducted with a very slow varying harmonic signal with a frequency $f_0 = 0.001Hz$ to determine if there was a loss in stiffness caused by damage during a previous test. Figure 4.18a displays the linearized stiffness of the VE material and Figure 4.18b displays the average stiffness for the control tests and the temperature of the VE material during the tests. Based on visual inspection of the specimens and the control tests it was determined that no damage had occurred other than small superficial tears in the VE material that had been sustained by the specimens when subjected to an excess of 1000 cycles at the design level strain.
4.2.4 Small-Scale Test Conclusions

The small-scale test results showed that the material properties are predictable and similar to the material properties summarized by the damper manufacturer. The material properties have strong dependency on the excitation frequency and VE material temperature, and to a lesser extent, the strain amplitude. The properties are stable up to the maximum strain recommended by NSEC of 400% and very stable and predictable at the targeted service level wind deformation amplitudes of 100% strain. Over long duration loadings, such as wind loading the VE material heats up and the $G_e$ and $\eta$ properties decrease, however the VE material temperature does not increase enough to cause significant drops in the VE material properties to affect the design. There were no signs of damage or drop in VE material stiffness or damping from one test to another at the design level strains.

The static stiffness of the VE material is less than the dynamic stiffness of the material and from the along-wind tests there is loss in the static stiffness over the duration of the test. This is further discussed in Chapter 6, outlining how to incorporate this in the design strategy.
4.3 FULL-SCALE FORK CONFIGURATION DAMPER TESTS AT ECOLE POLYTECHNIQUE MONTREAL (EPM)

Full-scale tests were conducted to validate the Fork Configuration Damper (FCD) system for use in real high-rise coupled wall buildings. The full-scale test design is based on the properties of typical high-rise structures and is targeted for an actual application in downtown Toronto. The VEM-111H material properties have been well characterized in small-scale tests conducted at the University of Toronto as well as material Nippon Steel in Japan, therefore the purpose of the full-scale FCD tests was to validate the overall system performance as expected based on the kinematic behavior of coupled wall systems, wall anchorage and damping material behavior during realistic wind and earthquake excitations. Also, the constructability of the FCD for use in coupled wall systems was studied in detail.

NSEC provided in-kind 6 full-scale specimens, 3-FCD-As and 3-FCD-Bs, which are shown in Figure 4.19.

![Figure 4.19 Full-Scale FCD Specimens](image)

4.3.1 Design of Test Specimen FCD-A

Test specimen FCD-A was designed for a high-rise building project in Toronto with the main design requirement of providing inherent viscous damping to the overall structure, while satisfying the stringent existing architectural requirements and the existing structural layout. The 85-storey residential high-rise building has a primary lateral load resisting system consisting of a reinforced concrete coupled wall system. The sample structure is described in detail in Section 6.5. The primary architectural design consideration was the depth of the FCD, which was kept to a minimum to maintain the structures current floor-to-ceiling height. Two-FCDs placed side-by-side were designed to replace a coupling beam of length 1,600 mm and a depth (including the slab) of 550 mm with a wall thickness of 800 mm. The tests at EPM consisted of 1-FCD-A cast between 450 mm thick walls.
For structures located in Toronto, structural engineers design concrete coupling beams to remain elastic for wind loads and to exhibit nominal ductility during earthquakes. Therefore the concrete coupling beams are designed for strength without ductile details. With these considerations in mind the FCD-A was designed to exhibit ductility levels larger than a typical concrete coupling beam designed in Toronto, however not as much ductility as FCD-B, which has a built-in structural “fuse” and is described in Section 4.3.2. The overall design process using FCD-A in tall buildings will be described in detail in Chapter 6. Figure 4.20 displays details of specimen FCD-A.

FCD-A consists of 17 layers of 330 wide by 520 long by 6.5\text{mm} thick viscoelastic material layers (VEM-111H) sandwiched between 16 layers of 9\text{mm} steel plates on the inside and two layers of 12\text{mm} steel plates on the outside. The steel plates are shot blasted and a primer was applied to the surfaces to increase the VEM-steel bond. The steel material properties used for design given by NSEC are displayed in Table 4-7.

**Figure 4.20 FCD-A Details**
The multiple 9 mm and 12 mm steel plates are bonded to the VE material and are sandwiched together with multiple filler plates and two built-up I-sections using 9 high-strength bolts tensioned to a minimum pre-tension force of $T = 379 kN$. The steel plate surfaces are blast-cleaned to achieve a slip coefficient of not less than $k_s = 0.50$, in order to maintain a slip critical connection under all expected test loading conditions. The bolt material properties used for design are shown in Table 4-8.

### TABLE 4-7 Steel Material Properties from NSEC: JIS G 3106 SM490A

<table>
<thead>
<tr>
<th>Plate Size</th>
<th>Yield Strength: Lower Limit (MPa)</th>
<th>Tensile Strength: Lower Limit (MPa)</th>
<th>Tensile Strength: Upper Limit (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plate Thickness Under 16 mm</td>
<td>325</td>
<td>490</td>
<td>610</td>
</tr>
<tr>
<td>Plate Thickness Over 16 mm</td>
<td>315</td>
<td>490</td>
<td>610</td>
</tr>
<tr>
<td>and Under 40 mm</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Plate Thickness Over 40 mm</td>
<td>295</td>
<td>490</td>
<td>610</td>
</tr>
</tbody>
</table>

The built-up I-sections have thick web sections, 22 mm, and relatively thin flange sections, 12 mm, with a 10 mm fillet weld connecting the web and flanges. The built-up I-sections sections have full penetration welds for both the web and flanges fixing them to a 50 mm thick end-plate; backing bars are utilized along the full length on the internal side of the web and below the top flange and above the bottom flanges. The weld material properties are shown in Table 4-9.

### TABLE 4-8 High Strength Bolt Material Properties from NSEC: JIS B 1186 F10T

<table>
<thead>
<tr>
<th>Bolt Size</th>
<th>Proof Stress: Minimum (MPa)</th>
<th>Tensile Strength: Lower Bound (MPa)</th>
<th>Tensile Strength: Upper Bound (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSTB M30 Bolts</td>
<td>900</td>
<td>1,000</td>
<td>1,200</td>
</tr>
</tbody>
</table>

### TABLE 4-9 Weld Properties from NSEC: AWS E70XX

<table>
<thead>
<tr>
<th>Weld Material</th>
<th>Specified Tensile Strength of Base Material (MPa)</th>
<th>Ultimate Strength of Weld Material (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AWS E70XX</td>
<td>490</td>
<td>560</td>
</tr>
</tbody>
</table>

Figure 4.21 shows some photos of the manufacturing process of the Fork Configuration Damper, including the VEM-steel bonding work and Figure 4.22 shows some photos of the complete FCD-A after shipment to EPM.
LENTON weldable half couplers were provided in-kind by ERICO, used to anchor the end-plate of the dampers into the concrete walls. The couplers are made with weldable steel, C1035A, formed with a “J” groove to facilitate a full-penetration weld to the structural plate with internal LENTON standard taper threads on the other side. The anchorage rebar is 35M, Grade 500, rebar 2,050 mm in length and were threaded by a local rebar supplier. To form the mechanical splice connection, the 35M rebar is torqued to 200 lb-ft to ensure a full mechanical connection.

Welding was performed by a professional welder using an electric arc weld with weld material E7018 at EPM. The weld material properties are displayed in Table 4-10 and the weld dimensions along with some photos of the welding application are shown in Figure 4.23.

Table 4-11 displays mill certificate test results of the LENTON weldable couplers (ERICO, 2009).
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

TABLE 4-10 Weld Properties from Lainco: AWS E7018

<table>
<thead>
<tr>
<th>Weld Material</th>
<th>Specified Tensile Strength of Base Material (MPa)</th>
<th>Ultimate Strength of Weld Material (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AWS E7018</td>
<td>490</td>
<td>560</td>
</tr>
</tbody>
</table>

TABLE 4-11 Mill Certificate Test Results from EL36C3J Lenton Couplers (ERICO, 2009)

<table>
<thead>
<tr>
<th>Lot No.</th>
<th>QTY.</th>
<th>Yield Strength (MPa)</th>
<th>Tensile Strength: (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L4812</td>
<td>54</td>
<td>649</td>
<td>693</td>
</tr>
<tr>
<td>L4961</td>
<td>210</td>
<td>624</td>
<td>636</td>
</tr>
</tbody>
</table>

Figure 4.22 FCD-A

a) 2-FCD-A

b) Built-up Steel I-Section

c) VEM Steel Layers
Fork Configuration Dampers (FCDs) for Enhanced Dynamic Performance of High-Rise Buildings

CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

4.3.1.1 Capacity Design of FCD-A

The thickness of the VE material layers was chosen to be 6.5\( mm \), which is the expected shear displacement under the Service Limit States in the across-wind 1 in 10 year wind storm for the case study structure. A capacity design was conducted to ensure that the failure mode of the damper was ductile and that unwanted or less predictable failure modes such as the VE material tearing or anchorage failure during extreme events are avoided.

Figure 4.23 Weldable Lenton Half Couplers End-Plate Connection
To evaluate the capacity design, a hierarchy of strength was established to ensure that a ductile and predictable failure mechanism governed the strength design of the assembly. A spreadsheet (see Appendix A) was created for the full FCD-A design, attempting to identify all appropriate failure mechanisms. For FCD-A the most reasonable and simplest failure mechanism was chosen to be flexural yielding. Evaluating the hierarchy of strength, the shear force is assumed to act at the center of the VE material with each built-up cantilever assembly undergoing single curvature as shown in Figure 4.24c. All moments and other failure mechanisms are related to the constant shear force, $F_{FCD}$, at the center inflection point.

**Figure 4.24 FCD-A: Capacity Design Force Definition**

The first seven hierarchy of strength mechanisms, evaluated and expressed as a shear force are described below:

1) $F_f = 525kN$ - The ULS, 1 in 50 year wind loading, factored design damper force in shear. All other element capacities must be higher than this.

2) $2\phi M_y/L_{CL} = 607kN$ - The factored yield resistance of the built-up assembly.

3) $2M_p/L_{CL} = 914kN$ - The plastic moment capacity of the build-up assembly in bending.

4) $2M_{pr}/L_{CL} = 1,105kN$ - The probable bending moment of the build-up assembly in bending assuming an increased material yield strength and overstrain in steel. This represents the minimum design strength that must be resisted with reasonable factors of safety for all mechanisms; the design load for all other connections is the force at which the probable moment occurs, i.e. $F'_f = 1,105kN$.

5) $2\phi M_{yEP}/L_{CL} = 1,112kN$ - The factored yielding of the end-plate in bending.

6) $2\phi M_{yAnc}/L_{CL} = 1,248kN$ - The factored anchorage connection resistance in bending.
7) \( V_r = 1,312 \text{kN} \) - The FCD shear force at which 5% probability of bolt slip occurs is considered to be the governing design force, because after slip occurs there are a number of other brittle mechanisms that may occur.

\[
F_{\text{FCD}} < 2 \phi M_y / L_{CL} < 2 M_p / L_{CL} < \frac{F_r}{2} = 2 M_{pr} / L_{CL} < 2 \phi M_{y,EP} / L_{CL} < 2 \phi (M_{y,anc} / L_{CL} < V_r)
\]

The calculated first seven FCD-A design strength items with the appropriate design clauses and formulae are shown in Table 4-12.

<table>
<thead>
<tr>
<th>Failure Mechanism</th>
<th>Governing Formula</th>
<th>Equation or Clause</th>
<th>FCD Shear Force ( F_{\text{FCD}} ) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factored 1 in 50 year Wind Load based on ETABS model</td>
<td>[ 1.4F_{\text{FCD}} = 1.4k_{\text{FCD}}u_{\text{FCD}} / \cos \theta ]</td>
<td>Eq. (3.6)</td>
<td>525</td>
</tr>
<tr>
<td>Factored Yield Resistance of Built-up Assembly in Bending</td>
<td>[ \phi M_y = \phi F_{\text{FCD}} S_x ]</td>
<td>CAN/CSA S16-01, Clause 13.5</td>
<td>607</td>
</tr>
<tr>
<td>Plastic Moment of the Built-up Assembly in Bending</td>
<td>[ M_p = F_{\text{y}} d^2 t_w / 4 + F_{\text{y}} b t_f (d + t_f) ]</td>
<td>CAN/CSA S16-01, Clause 13.5</td>
<td>914</td>
</tr>
<tr>
<td>Probable Moment of the Built-up Assembly in Bending</td>
<td>[ M_{pr} = 1.1 R_y M_{pr} ]</td>
<td>CAN/CSA S16-01, Clause 27.2.2</td>
<td>1,105</td>
</tr>
<tr>
<td>Factored Yield Strength of End-plate</td>
<td>[ \phi M_{y,EP} = \phi F_{\text{y,EP}} S_{x,EP} ]</td>
<td>CAN/CSA S16-01, Clause 13.5</td>
<td>1,112</td>
</tr>
<tr>
<td>Factored Yield Strength of Anchorage</td>
<td>[ \phi M_{y,anc} = \phi \sum A_v E_v d(c - d) + \phi \beta_1 \alpha_1 f_s A_v (c - a/2) ]</td>
<td>CSA A23.3-04, Clause 10.1</td>
<td>1,248</td>
</tr>
<tr>
<td>Factored Slip Resistance of the Bolted Connection</td>
<td>[ V_r = C \left(0.53 c_1 k_{nm} a_p F_{\text{y}} \right) ]</td>
<td>CAN/CSA S16-01, Clause 13.12.1.2 and Spreadsheet Method, Crawford and Kulak (1971)</td>
<td>1,320</td>
</tr>
</tbody>
</table>

### 4.3.2 Design of Test Specimen FCD-B

Test Specimen FCD-B was designed for a 50-storey high-rise building in downtown Vancouver, to replace a 750\( \text{mm} \) deep, 400\( \text{mm} \) wide and 2,100\( \text{mm} \) long RC coupling beam or alternatively two dampers would replace a 750\( \text{mm} \) deep, 800\( \text{mm} \) wide and 2,100\( \text{mm} \) long RC coupling beam. In more earthquake prone areas structural engineers design RC coupling beams to remain elastic for wind loads and exhibit ductility during strong earthquakes acting as a “fuse” mechanism designed using a capacity design methodology. With these design requirements in mind, FCD-B was designed to include reduced beam section (RBS) structural “fuse” mechanisms. Figure 4.25 gives details of the FCD-B specimen.

FCD-B consists of 15 layers of 380 wide by 520 long by 6.5\( \text{mm} \) thick viscoelastic material (VEM 111H) sandwiched between 16 layers of 9\( \text{mm} \) steel plates on the inside and two layers of 12\( \text{mm} \) steel plates on the outsides. The VEM-steel bonding, Lenton coupler welding procedure and high-strength bolting procedure is the same as for FCD-A and the steel and bolt material properties are the same.
The RBSs have 12 mm thick web sections and 25 mm thick flanges, with a 10 mm fillet weld connecting the web and the flanges. The built-up sections have full penetration welds for both the web and flanges fixing them to the 50 mm thick end-plate. Backing bars are utilized along the full length on the internal side of the web and below and above the top and bottom flanges, respectively.

4.3.2.1 Capacity Design of FCD-B

The RBS concept was first introduced for moment frames by Chen and Yeh (1994) and gained popularity after large numbers of weld failures were observed in steel moment frames during the 1994 Northridge Earthquake. In the RBS, top and bottom flanges are reduced to localize flexural yielding at a prescribed distance away from the end connection, preventing premature weld fracture at the end connection and decreasing the moment demand introduced to the beam-column connection interface.

Figure 4.25 FCD-B Details
Past tests have shown that after some plastification occurs in the Reduced Beam Section, the webs tend to buckle out-of-plane as well as local buckling in the flange, resulting in substantial reduction in strength and stiffness and degradation of hysteretic properties. To address this, Naeim et al. (2001) introduced a beam-web-stiffener detail to postpone the post-yield buckling of the webs in large beams used in Special Moment Resisting Frames (SMRFs) for tall buildings. Large full-scale cyclic beam sections with beam-web-stiffeners were tested at the University of Nevada, Reno, that exhibited reliable performance with high ductility and high stiffness (Naeim et al. 2001).

a) RBS Definitions

b) RBS for Tall Buildings (adapted from Naeim et al. 2001)

![Figure 4.26 RBS Details Incorporated in FCD-B Design](image)

A similar detail with a 12\(\text{mm}\) thick, 30\(\text{mm}\) wide and 330\(\text{mm}\) long plate welded to the middle of the web section using 6\(\text{mm}\) fillet welds along the length on the two outer web plates was used in the design of FCD-B. The RBS was designed according to guidelines suggested for Moment Connections for Seismic Applications (CISC 2005) and American guidelines (ANSI/AISC 358-05 2006 and FEMA 2000).

The probable peak plastic moment capacity, \(M_{pr}\), at the location of the plastic hinge is calculated using the CISC (2005) guideline as:

\[
M_{pr} = C_{pr} R_y F_y Z_e
\]

where \(Z_e\) is the effective plastic modulus of the beam section at the reduced beam section, \(R_y F_y\) is the expected yield stress as defined in Clause 27.1.7 of S16-01 and \(C_{pr}\) is a factor accounting for the effects of strain hardening and local restraint calculated as:

\[
C_{pr} = \frac{F_y + F_u}{2F_y}
\]

The 25\(\text{mm}\) thick and 100\(\text{mm}\) wide flange sections have a cut section of \(c = 25\text{mm}\) with a radius of \(r = 463\text{mm}\) and a chord length of \(s = 300\text{mm}\) forming a flange width at the reduced section of \(b' = 50\text{mm}\). The RBS chord length was chosen as \(s = 300\text{mm}\) to ensure the RBS location is away from
the bolted section. The probable plastic moment at the center of the chord length calculated using Eq. (4.16) is \( M_{pr} = 523kNm \), which occurs at a FCD shear force of \( F_{pr} = 1230kN \). The connection at the end-plate is precluded if \( 2M_{prBS}/L_{RBS} < 2\phi M_{pCL}/L_{CL} \).

Using the same procedure as in the design of FCD-A, a hierarchy of strength is again evaluated; the shear force is assumed to act at the center of the VE material with each built-up cantilever assembly undergoing single curvature as shown in Figure 4.27c. All moments and other failure mechanisms are related to the constant shear force, \( F_{FCD} \), at the center inflection point.

1) \( F_f = 525kN \) - The ULS, 1 in 50 year wind loading, factored design damper force in shear. All other element capacity must be higher than this.
2) \( 2\phi M_{yRBS}/L_{RBS} = 600kN \) - The factored yield resistance of the RBS built-up assembly at the center of the RBS.
3) \( 2\phi M_{yCL}/L_{CL} = 784kN \) - The factored yield resistance of the built-up assembly at the end-plate.
4) \( 2M_{pRBS}/L_{RBS} = 875kN \) - The plastic moment capacity of the RBS assembly in bending.
5) \( 2\phi M_{pCL}/L_{CL} = 1,057kN \) - The plastic moment capacity of the built-up assembly at the end-plate. Note this pre qualifies the connection i.e. \( 2M_{pRBS}/L_{RBS} < 2\phi M_{pCL}/L_{CL} \).

---

**Figure 4.27 FCD-B: Capacity Design Force Definition**

The first eight hierarchy of strength mechanisms, evaluated and expressed as a shear force are described below:

1) \( F_f = 525kN \) - The ULS, 1 in 50 year wind loading, factored design damper force in shear. All other element capacity must be higher than this.

2) \( 2\phi M_{yRBS}/L_{RBS} = 600kN \) - The factored yield resistance of the RBS built-up assembly at the center of the RBS.

3) \( 2\phi M_{yCL}/L_{CL} = 784kN \) - The factored yield resistance of the built-up assembly at the end-plate.

4) \( 2M_{pRBS}/L_{RBS} = 875kN \) - The plastic moment capacity of the RBS assembly in bending.

5) \( 2\phi M_{pCL}/L_{CL} = 1,057kN \) - The plastic moment capacity of the built-up assembly at the end-plate. Note this pre qualifies the connection i.e. \( 2M_{pRBS}/L_{RBS} < 2\phi M_{pCL}/L_{CL} \).
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

6) \(2 \phi M_y_{Anc}/L_{CL} = 1,129 \text{ kN}\) - The factored anchorage connection resistance in bending. Note this is deemed acceptable since the unfactored connection resistance is significantly more than the factored probable bending moment.

7) \(2 M_{prRBS}/L_{RBS} = 1,230 \text{ kN}\) - The probable bending moment of the build-up assembly in bending assuming an increased material yield strength and overstrain in steel. This represents the minimum design strength that must be resisted with a reasonable factor of safety for all mechanisms; the design load for all other connections is the force at which the probable moment occurs, i.e. \(F_p' = 1,230 \text{ kN}\).

8) \(V_r = 1,295 \text{ kN}\) - The FCD shear force at which 5% probability of slip occurs is considered to be the governing design force, because after slip occurs there are a number of other brittle mechanisms that may occur.

\[ F_{FCDF} < 2 \phi M_y_{RBS}/L_{RBS} < 2 \phi M_y_{CL}/L_{CL} < 2 M_{prRBS}/L_{RBS} < 2 \phi M_y_{ Anc}/L_{CL} < F_p' = \frac{2 M_{prRBS}}{L_{RBS}} < V_r \]

The calculated first eight FCD-B design strength items with the appropriate clauses and formulas are located in Table 4-13. Figure 4.28 displays some photos of damper FCD-B and the RBS. The calculated FCD-B design strength items with the appropriate clauses and formulas are located in Table 4-13.

<table>
<thead>
<tr>
<th>Failure Mechanism</th>
<th>Governing Formula</th>
<th>Equation or Clause</th>
<th>FCD Shear Force (F_{FCDF}(\text{kN}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factored 1 in 50 year Wind Load based on ETABS model</td>
<td>(1.4 F_{FCDF} = 1.4k F_{FCDF} \cdot 10^0 / \cos \theta)</td>
<td>Eq. (3.6)</td>
<td>525</td>
</tr>
<tr>
<td>Factored Yield Resistance at the Built-up RBS in Bending</td>
<td>(\phi M_y_{RBS} = \phi F_y S_i_{RBS})</td>
<td>CAN/CSA S16-01, Clause 13.5</td>
<td>600</td>
</tr>
<tr>
<td>Factored Yield Resistance at the End-plate Section in Bending</td>
<td>(\phi M_y_{CL} = \phi F_y S_i_{EP})</td>
<td>CAN/CSA S16-01, Clause 13.5</td>
<td>784</td>
</tr>
<tr>
<td>Plastic Moment at the Built-up RBS in Bending</td>
<td>(M_{prRBS} = F_{yR} d^2 t_w/4 + F_{yF} b' t_f (d + t_f))</td>
<td>CAN/CSA S16-01, Clause 13.5</td>
<td>875</td>
</tr>
<tr>
<td>Plastic Moment at the End-plate sections in Bending</td>
<td>(M_{prCL} = F_{yR} d^2 t_w/4 + F_{yF} b' t_f (d + t_f))</td>
<td>CAN/CSA S16-01, Clause 13.5</td>
<td>1,057</td>
</tr>
<tr>
<td>Factored Yield Strength of Anchorage</td>
<td>(\phi M_y_{Anc} = \phi_x \Sigma A_i E_i d (c - d) + \phi_x \beta_j \alpha_i s_i') A_i (c - a/2))</td>
<td>CSA A23.3-04, Clause 10.1</td>
<td>1,129</td>
</tr>
<tr>
<td>Probable Moment of the Built-up RBS</td>
<td>(M_{prRBS} = C_p R_s M_{prRBS})</td>
<td>CISC, 2004 guideline</td>
<td>1,230</td>
</tr>
<tr>
<td>Factored Slip Resistance of Bolted Connection</td>
<td>(V_r = C(0.53 c_1 k_m A_m F_u))</td>
<td>CAN/CSA S16-01, Clause 13.12.1.2 and Spreadsheet Method, Krawford and Kulak (1971)</td>
<td>1,295</td>
</tr>
</tbody>
</table>
4.3.3 Full-Scale Test Design

The purpose of the full-scale tests was to validate the Fork Configuration Damper (FCD) in a configuration representative of how the damper would be implemented in a real coupled wall high-rise building. The design of the full-scale wall system was based on the properties of the sample structure in downtown Toronto. The VEM-111H material properties were well characterized in the small-scale tests conducted at the University of Toronto in 2007 as well as tests from the damper manufacturer, therefore the purpose of the full-scale FCD tests was to validate the overall system performance as expected based on the kinematic behavior of reinforced concrete coupled wall systems, wall anchorage and damping material behavior during realistic wind and earthquake excitations.

![2-FCD-B](image1)

b) RBS  
c) RBS to End-Plate Connection

Figure 4.28 FCD-B
Under lateral vibrations the FCDs will behave primarily in shear caused by the large concrete walls rotating about their centerlines. A racking test was conducted at EPM whereby two walls are coupled with the FCD and two stiff channel sections in parallel connected to the walls via pin connections at the actuator height. The walls are connected to base pins and when actuator displacements are applied the walls rotate at the base pins engaging the FCD in shear through a racking movement. Since the walls are connected by a stiff axial pin connection the walls move the same amount laterally mimicking the rigid diaphragm behaviour of a floor slab. Figures 4.29a and 4.30a show the application of the FCD specimens in the case study structures and Figures 4.29c and 4.30c display the full-scale test configuration for FCD-A and B, respectively. Figures 4.29b and 4.30b and Figures 4.29d and 4.30d display the exaggerated deformed shape under lateral loads of the real application and the test setup for dampers FCD-A and FCD-B, respectively. The walls in the test setup are not required on the outside of the base pin centerline, because of the direct force transfer mechanism to the pins.

The test setup consisted of two pins post-tensioned to the strong floor at distances of 5,500 mm and 6,000 mm for dampers A and B, respectively (6,000 mm and 6,500 mm to the outside wall edges) and two actuators fixed to the strong wall connected to the full-scale setup at a height of 4,087 mm above the centerline of the pins. There were two precast wall assemblies (left and right), each with 3 individual panels post-tensioned together. The precast wall elements were 450 mm thick with embedded PT-ducts, which
a) Case Study FCD Application

Fork Configuration Dampers (FCDs) for Enhanced Dynamic Performance of High-Rise Buildings

b) Exaggerated Deformed Shape

Figure 4.30 Full-Scale FCD Test Deformed Shape, FCD-B

The 28 day concrete strength used for all pours was $f'_c = 60 MPa$. The walls were designed to remain uncracked through the maximum lateral load of the actuators with a factor of safety. Multiple finite element and strut and tie models were utilized to determine the flow of forces in the concrete wall elements as well as to design the reinforcement. All drawings of the test setup including the steel elements built by CanAM in Boucherville Quebec as well as the precast wall pieces built by Béton Brunet in Valleyfield, Quebec are located in Appendix B.
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

Figure 4.31 Full Scale FCD Test Setup, FCD-A
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

Figure 4.32 Full Scale FCD Test Setup, FCD-B
The two 1000 kN actuators were bolted to stiff steel connection arms that were used to transfer the load to a W510x196 application beam anchored by a slip-critical bolted connection. The W-section was cast into the concrete using shear studs and welded rebar. There was another W510x196 beam section cast in the left hand wall connecting the left and right walls together by 4” steel pins and two stiff MC460x86 channel sections (for both FCD-A and FCD-B sizes). The W-sections had holes for the Dywidag threadbars with two stiffeners between the top and bottom flanges where the post-tensioning is applied. Figure 4.33a show the crack control rebar that was welded to the top load application beams.

![Figure 4.33a: Welding Anchorage Rebar to Steel Precast Elements](image)

The two bottom wall segments have fabricated steel elements where the base pin hinge connects to the specimen, which was then anchored to the concrete with anchor studs and welded rebar in the areas of high tensile stress above the pins. The steel section had holes to allow for the post-tensioning bars to pass through and be anchored at the bottom. Figures 4.33b and 4.33c show the welded rebar in the areas of high tensile stress above the base pins.

![Figure 4.33b: Base Pin Rebar Welds](image)

![Figure 4.33c: Bottom Pin Rebar Welds](image)

The two bottom wall segments have fabricated steel elements where the base pin hinge connects to the specimen, which was then anchored to the concrete with anchor studs and welded rebar in the areas of high tensile stress above the pins. The steel section had holes to allow for the post-tensioning bars to pass through and be anchored at the bottom. Figures 4.33b and 4.33c show the welded rebar in the areas of high tensile stress above the base pins.

The top and bottom walls and the axial sections were used for all tests and the middle walls and FCD were removed after each test and replaced by the next specimen. The concrete walls were designed as sufficiently rigid to engage the FCD in shear through the combined racking motion of the two walls.

A match cast technique was used to build the wall specimens to ensure the correct final location in order to satisfy the geometry of the wall specimens, actuators and pins at the structures laboratory at EPM. The precast operations were performed on a 24’ by 16’ flat steel casting deck. Some photos from the construction of the top and bottom match cast template walls are shown in Figure 4.34. Figures 4.35 and 4.36 show photos of the FCD-A and FCD-B specimen pours, respectively. Figure 4.37 shows pictures of the erection process that was used to build the test setup at EPM.
To provide lateral support to the system top and bottom lateral supports were installed as illustrated in Figure 4.38. Two-HSS254x254x6.4, eight meters in length are bolted to 3-254x310 W-section columns anchored to the strong wall. Four greased strips of teflon-like plastic that were fixed to the bottom wall sections and another four greased strips were placed on the inside of the lateral support HSSs where the concrete meets the lateral supports. Shims were used to build up any space between the teflon-like sheets and ensure contact between the walls and lateral supports. The top lateral supports consisted of two built-up braces which were attached to the top walls with angles cast into the concrete. The braces travelled between HSS guides connected to the wall allowing the braces to travel with the walls, on the same greased teflon-like surfaces.

After the FCD-concrete wall specimens were tested to failure the actuator was disconnected from the top right wall, the post-tensioning was propped up keeping them in place and all post-tensioning was released. The two top walls connected by the stiff channel section were lifted vertically using two 5tonne cranes keeping the top walls from rotating. The FCD-concrete wall specimen was removed and the next specimen was brought into place. The assembly process was repeated for all tested specimens.
4.3.4 Test Instrumentation

The instrumentation can be divided into two types; displacement instrumentation, which includes string potentiometers, lvdt's and clinometers, and strain gauge instrumentation. The instrumentation was used to determine two main actions; local FCD response and overall test kinematics. There were also two video cameras; one capturing the overall kinematic response as well as one capturing the local FCD response. Also, a non-contact temperature infracam was rented for some of the FCD-A2 tests and for all FCD-B tests to capture the temperature change in the VE material. A summary of the instrumentation that was used for all tests is located in Figure 4.39 and Table 4-14.

In order to measure the local FCD deformations, two stiff aluminum braces were cantilevered off the face of each wall, with 5 string potentiometers (2x(S1-S5)) each fixed to the aluminum braces measuring local displacements along the length of the FCD. Through the string potentiometers measurements the FCD shear displacement, VE material displacement, chord rotations and losses in the connections were determined. A more thorough discussion on using these string potentiometers for determining FCD deformations is provided in Section 4.3.7.2.
The overall wall kinematics were monitored using four string potentiometers (S8 to S11), two measuring wall horizontal displacements at two points along the height of the right wall (S8 and S9) and two measuring vertical displacements (S10 and S11) at the bottom edge of the two walls. Two clinometers (C1 and C2) were attached to the back of the concrete walls to measure the rotations.

Two small LVDTs (2xL1) were attached at the end of each aluminum brace. They extended to a teflon-like sheet that was attached to the concrete walls monitoring FCD axial displacements. To monitor the FCD end-plate uplift from the concrete walls, two small LVDTs (2xL2) were attached to stiff aluminum angles horizontally, which were then attached to the aluminum angles and were cantilevered in front of the end-plates to measure the uplift of the FCD end-plates, relative to the wall. Two Small LVDTs were used to measure if there was slip of the two walls relative to one another on the right wall (L3 and L4) and an LVDT to measure if there was any slip at the actuator connection (L5). Four linear pots (P1-P4) were used to measure the base pin movements (2 vertical and 2 horizontal) and two linear pots (P5 and P6) were used to measure the pin movement in the two top pins.

Figure 4.36 FCD-B Specimen Pour

a) Concrete Set FCD-A  b) Moving Bottom Walls  c) Moving Specimen

d) HSSs for Damper Movement  e) New Layout for FCD-B Pours  f) End Zone Reinforcement

g) End of Pour  h) Concrete Set  i) Specimen Removal
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

Figure 4.37 Setup at EPM

Figure 4.38 Lateral Support
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

S1 S2 S3 S4 L2
S1

DETAIL A
S6 S7

L4

DETAIL A
L3

S8

2-String Pots

DETAIL B
S12 S13

S14 S15

P1 P2

P4

P5

P6

P3

S10

S9

2-Linear Pots, 1 horizontal at each pin

2-Aluminum arms cantilevered from each wall

4-String Pots Out-of-Plane

2-String Pots

Figure 4.39 Instrumentation Map

FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

2-Aluminum arms cantilevered from each wall

2-String Pots Out-of-Plane

6 - Linear Strain Gauges (3 Front and 3 Rear)

8-Rosette Gauges (4 Front and 4 Rear)

8-Linear Gauges (4 Front and 4 Rear)

B-Rosette Gauges (4 Front and 4 Rear)

2-Linear Pots (1 Front and 1 Rear)

2-Linear Pots (1 Front and 1 Rear)

2-Linear Pots, 1 vertical and 1 horizontal at each base pin

6 - Linear Strain Gauges (3 Front and 3 Rear)

8-Rosette Gauges (4 Front and 4 Rear)

8-Linear Gauges (4 Front and 4 Rear)

B-Rosette Gauges (4 Front and 4 Rear)

2-Linear Pots (1 Front and 1 Rear)

2-Linear Pots (1 Front and 1 Rear)
The majority of strain gauges were placed on the FCD specimens with some additional strain gauges on the axial channel sections. There were 16-unaxial strain gauges (SG1-SG16) for FCD-A on the top and bottom of the built-up I-sections, measuring uni-axial strains at 50mm away from the end-plate. For the FCD-B specimen there were an additional 4 linear gauges at the minimum width of the RBS section at 200mm away from the end-plates. There were a total of 8-rosettes strain gauges (SG17-S4G1) on the front and back faces of the built-up I-section for both FCD-A and B specimens at one third of the built-up I-section height, 50mm away from the end-plates measuring shear strain. Each axial section had 3 uniaxial strain gauges to measure the linear strain across the channel sections.

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Instrumentation Type</th>
<th>Label</th>
<th>Number of Instruments</th>
</tr>
</thead>
<tbody>
<tr>
<td>FCD Relative Displacements (FCD and VE Material Shear Displacements, Chord Rotations and Elastic Losses)</td>
<td>String Potentiometers</td>
<td>2x(S1-S5)</td>
<td>10</td>
</tr>
<tr>
<td>FCD Shear Displacements</td>
<td>String Potentiometers</td>
<td>S6-S7</td>
<td>2</td>
</tr>
<tr>
<td>Overall Wall Displacements and Rotations and Test Kinematics</td>
<td>String Potentiometers</td>
<td>S8-S11</td>
<td>4</td>
</tr>
<tr>
<td>Out-of-Plain Wall Movement</td>
<td>String Potentiometers</td>
<td>S12-S15</td>
<td>4</td>
</tr>
<tr>
<td>Axial FCD Displacements</td>
<td>LVDTs</td>
<td>2xL1</td>
<td>2</td>
</tr>
<tr>
<td>FCD End-plate Uplift</td>
<td>LVDTs</td>
<td>2xL2</td>
<td>2</td>
</tr>
<tr>
<td>Slip Critical Connection Plate Slippage</td>
<td>LVDTs</td>
<td>L3-L4</td>
<td>2</td>
</tr>
<tr>
<td>Slip of Actuator Connection</td>
<td>LVDTs</td>
<td>2xL5</td>
<td>1</td>
</tr>
<tr>
<td>Bottom Pin Slack</td>
<td>Linear Pots</td>
<td>P1-P4</td>
<td>4</td>
</tr>
<tr>
<td>Top Pin Slack</td>
<td>Linear Pots</td>
<td>P5-P6</td>
<td>2</td>
</tr>
<tr>
<td>FCD Uniaxial Strains at the top of the I-sections</td>
<td>Uniaxial Strain Gauges</td>
<td>SG1-SG16</td>
<td>16</td>
</tr>
<tr>
<td>FCD Shear Strains at the webs of the I-sections</td>
<td>Rosette Strain Gauges</td>
<td>SG17-S4G1</td>
<td>24</td>
</tr>
<tr>
<td>Axial Section Strains and Force</td>
<td>Uniaxial Strain Gauges</td>
<td>SG42-SG47</td>
<td>6</td>
</tr>
</tbody>
</table>
4.3.5 Material Tests

4.3.5.1 Coupon Tests Cut from FCD A-2

To independently determine the material properties of the Japanese grade steel, FCD A-2 was flame cut after the tests to separate it from the concrete block and shipped to a machine shop where 55 mm wide coupons were cut at the undamaged bolted built-up section, shown in Figure 4.40a. All steel was grade JIS G 3106 SM490A except for the 22 mm filler plates, which were a lower grade steel, JIS G 3101 SS400, because they were only expected to have clamping stresses if the connection remained slip-critical. The tensile coupon geometry is shown in Figure 4.40b. All coupons were cut to ASTM International Standards (ASTM 2009) geometry, while the two higher strength steel 22 mm web coupons were modified to 35 mm to decrease the ultimate force of the coupon.

Table 4-15 and Figures 4.41a, b and c show the grade JIS G 3106 SM490A steel plate tensile coupon results and Table 4-16 and Figure 4.41d show the grade JIS G 3101 SS400 steel plate tensile coupon test results.

4.3.5.1.1 Concrete Cylinder Tests

Concrete with a 28 day design strength of $f_c' = 60 MPa$ was used for all concrete pours. For each pour three concrete cylinders were cast. Table 4-17 lists the concrete cylinder compression strengths for all walls at 7 days and for two walls at 28 days, performed at the precasters quality control department.
4.3.6 Test Summary and Loading Protocol

The displacements of the two actuators were slaved and were controlled using a preprogrammed displacement signal. Design strains were targeted in the VE material, \( \gamma_{VE} \), however it was difficult to obtain the exact design strain throughout the test duration, because of base pin and channel pin slack, elastic losses in the system as well as degradation of stiffness properties during the tests.

The testing program consisted of four different types of tests, which are listed below:

1) FCD-VE harmonic property characterization at expected building frequencies
2) Representative FCD time-histories to establish the FCD behaviour during actual earthquake and wind events.
3) Ultimate dynamic tests to establish large amplitude dynamic characteristics.
4) Quasi-static ultimate static tests to establish the ultimate FCD strength.

Table 4-18 gives a summary of all the tests that were conducted for all specimens.
### TABLE 4-15 Tensile Coupon Results (JIS G 3106 SM490A)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Modulus of Elasticity, $E$ (GPa)</th>
<th>Yield Strength, $F_y$ (MPa)</th>
<th>Ultimate Strength, $F_u$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9-1</td>
<td>205</td>
<td>383</td>
<td>554</td>
</tr>
<tr>
<td>9-2</td>
<td>211</td>
<td>366</td>
<td>549</td>
</tr>
<tr>
<td>9-3</td>
<td>224</td>
<td>367</td>
<td>539</td>
</tr>
<tr>
<td>9 mm Average</td>
<td>213</td>
<td>372</td>
<td>547</td>
</tr>
<tr>
<td>12 mm Outer Plate</td>
<td>201</td>
<td>353</td>
<td>537</td>
</tr>
<tr>
<td>22 mm Vertical I-Section Plates</td>
<td>198.4</td>
<td>327</td>
<td>517</td>
</tr>
<tr>
<td>22-1</td>
<td>199.7</td>
<td>327</td>
<td>515</td>
</tr>
<tr>
<td>22-2</td>
<td>204.9</td>
<td>327</td>
<td>516</td>
</tr>
<tr>
<td>22 mm Average</td>
<td>202</td>
<td>327</td>
<td>516</td>
</tr>
</tbody>
</table>

### TABLE 4-16 Tensile Coupon Results of (JIS G 3101 SS400)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Modulus of Elasticity, $E$ (GPa)</th>
<th>Yield Stress, $F_y$ (MPa)</th>
<th>Ultimate Strength, $F_u$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>22-3</td>
<td>193.9</td>
<td>253</td>
<td>421</td>
</tr>
<tr>
<td>22-4</td>
<td>199.7</td>
<td>250</td>
<td>419</td>
</tr>
<tr>
<td>22-5</td>
<td>204.9</td>
<td>250</td>
<td>422</td>
</tr>
<tr>
<td>Average</td>
<td>199.5</td>
<td>251</td>
<td>421</td>
</tr>
</tbody>
</table>

### TABLE 4-17 Concrete Cylinder Compressive Strength

<table>
<thead>
<tr>
<th>Date</th>
<th>Pour</th>
<th>7 day - 1</th>
<th>28 day - 1</th>
<th>28 day - 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.29.2009</td>
<td>Match Cast Walls (Top and Bottom Right and Left)</td>
<td>53.8</td>
<td>72.4</td>
<td>72</td>
</tr>
<tr>
<td>6.09.2010</td>
<td>FCD A-1</td>
<td>40.6</td>
<td>60.9</td>
<td>61.2</td>
</tr>
<tr>
<td>6.16.2010</td>
<td>FCD A-2</td>
<td>54.6</td>
<td>73.4</td>
<td>73</td>
</tr>
<tr>
<td>6.19.2010</td>
<td>FCD A-3</td>
<td>59.4</td>
<td>71.4</td>
<td>71.7</td>
</tr>
<tr>
<td>6.24.2010</td>
<td>FCD B-1</td>
<td>49.9</td>
<td>60.0</td>
<td>60.3</td>
</tr>
<tr>
<td>6.29.2010</td>
<td>FCD B-2</td>
<td>52.0</td>
<td>68.5</td>
<td>68.3</td>
</tr>
<tr>
<td>7.01.2010</td>
<td>FCD B-3</td>
<td>47.3</td>
<td>72.5</td>
<td>73</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>51.1</td>
<td>68.4</td>
<td></td>
</tr>
</tbody>
</table>
### TABLE 4-18 Full-scale Loading Protocol

<table>
<thead>
<tr>
<th>FCD</th>
<th>Test Set #1</th>
<th>Test Set #2</th>
<th>Test Set #3</th>
<th>Test Set #4</th>
<th>Test Set #5</th>
<th>Test Set #6</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Working Strain Characterization</td>
<td>Higher Strain Characterization</td>
<td>Ultimate Strain Characterization</td>
<td>Real Wind and Earthquake Time Histories</td>
<td>Ultimate Quasi-Static Cyclic Loading</td>
<td>Ultimate Dynamic</td>
</tr>
<tr>
<td>A-1</td>
<td>$\gamma_{VE0} = \pm 50%$ at $f_0 = 0.1, 0.3, 0.5Hz$ for 500 cycles</td>
<td>$\gamma_{VE0} = \pm 100%$ at $f_0 = 0.1, 0.2, 0.3Hz$ for 100 cycles</td>
<td>$\gamma_{VE0} = \pm 200%$ at $f_0 = 0.1, 0.2, 0.3Hz$ for 10 cycles</td>
<td>$u_{FCD0} = \pm 23mm$ at $f_0 = 0.3Hz$, displacement increased by $u_{FCD0} = \pm 6.5mm$ for 10 cycles until failure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A-2</td>
<td>Working Strain Characterization</td>
<td>Higher Strain Characterization</td>
<td>Ultimate Strain Characterization</td>
<td>Real Wind and Earthquake DLE and MCE Time Histories</td>
<td>Ultimate Dynamic $u_{FCD0} = \pm 19mm$ at $f_0 = 0.3Hz$, displacement increased by $u_{FCD0} = \pm 6.5mm$ for 10 cycles until failure</td>
<td></td>
</tr>
<tr>
<td>A-3</td>
<td>$\gamma_{VE0} = \pm 50%$ at $f_0 = 0.1, 0.3, 0.5Hz$ for 500 cycles</td>
<td>Higher Strain Characterization</td>
<td>Ultimate Strain Characterization</td>
<td>$u_{FCD0} = \pm 46mm$ and $u_{FCD0} = \pm 52mm$ at $f_0 = 0.2Hz$ for 10 cycles</td>
<td>Ultimate Quasi-Static Cyclic Loading</td>
<td></td>
</tr>
<tr>
<td>B-1</td>
<td>Working Strain Characterization</td>
<td>Higher Strain Characterization</td>
<td>Ultimate Strain Characterization</td>
<td>Real Wind and Earthquake Time Histories</td>
<td>Ultimate Dynamic $u_{FCD0} = \pm 26mm$ at $f_0 = 0.3Hz$, displacement increased by $u_{FCD0} = \pm 6.5mm$ for 3 cycles</td>
<td></td>
</tr>
<tr>
<td>B-2</td>
<td>$\gamma_{VE0} = \pm 50%$ at $f_0 = 0.1, 0.3, 0.5Hz$ for 500 cycles</td>
<td>Real Wind and Earthquake Time Histories</td>
<td>Ultimate Strain Characterization</td>
<td>Real Wind and Earthquake Time Histories</td>
<td>Ultimate Quasi-Static Cyclic Loading</td>
<td></td>
</tr>
<tr>
<td>B-3</td>
<td>Working Strain Characterization</td>
<td>Higher Strain Characterization</td>
<td>Ultimate Strain Characterization</td>
<td>Real Wind and Earthquake Time Histories</td>
<td>Real Wind and Earthquake Time Histories</td>
<td>Ultimate Quasi-Static Cyclic Loading</td>
</tr>
</tbody>
</table>

**Fork Configuration Dampers (FCDs) for Enhanced Dynamic Performance of High-Rise Buildings**
4.3.6.1 Harmonic Characterization

The harmonic loading protocols were applied with a displacement control signal and scaled linearly through trial and error to achieve the targeted harmonic peak strain.

4.3.6.1.1 Working Strain Harmonic Characterization

The working strain harmonic characterization tests were based on typical wind level FCD working strain conditions. All specimens were subjected to sinuisoidal low actuator amplitude displacements producing VE material strains of $\gamma_{VE0} = \pm 50\%$ for frequencies $f_0 = 0.1, 0.3, 0.5Hz$ for 500 cycles. This subset of non-destructive loading conditions was repeated for each of the damper specimens before applying any of the other loading protocols.

4.3.6.1.2 Higher Strain Harmonic Characterization

FCD-A1, FCD-A2, FCD-B1 and FCD-B3 were subjected to sinuisoidal actuator displacements producing VE material strains of $\gamma_{VE0} = \pm 100\%$ for frequencies $f_0 = 0.1, 0.2, 0.3Hz$ for 100 cycles.

4.3.6.1.3 Ultimate Strain Harmonic Characterization

FCD-A1, FCD-A2, FCD-B1 and FCD-B3 were subjected to sinuisoidal actuator displacements producing VE material strains of $\gamma_0 = \pm 150, 200\%$ for frequencies $f_0 = 0.1, 0.2, 0.3Hz$ for 10 cycles.

4.3.6.2 FCD Wind and Earthquake Time-Histories

FCD response time-histories were analytically obtained from FE models of the 85-storey case study from wind load time-histories and multiple large earthquake time-histories. These results from the time-history analyses were applied as the actuator displacement controlled signal using scaling to achieve the design level strains. The design based max strains used in the tests are listed in Table 4-19. These strains were then increased by 1.5 and 2 times to evaluate the FCD response at larger strains.

<table>
<thead>
<tr>
<th>Across SLS</th>
<th>Across ULS</th>
<th>Along SLS</th>
<th>Along ULS</th>
<th>Design Based Earthquake</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_{VEDynamic}$</td>
<td>$\gamma_{VEDynamic}$</td>
<td>$\gamma_{VEMean} \pm \gamma_{VEDynamic}$</td>
<td>$\gamma_{VEMean} \pm \gamma_{VEDynamic}$</td>
<td>$\gamma_{VEDynamic}$</td>
</tr>
<tr>
<td>$\gamma_{VEMax} (%)$</td>
<td>$\pm 100$</td>
<td>$\pm 135$</td>
<td>$75 \pm 50$</td>
<td>$100 \pm 70$</td>
</tr>
</tbody>
</table>
4.3.6.3 Ultimate Tests

4.3.6.3.1 Dynamic Ultimate Tests

To test the ultimate capacity of the damper dynamically, specimens FCD-A2, FCD-A3, FCD-B2 and FCD-B3 were subjected to harmonic sinusoidal loading at frequencies of $f_0 = 0.2\, \text{Hz}$ and $f_0 = 0.3\, \text{Hz}$ for multiple cycles. The tests were started at approximately $\gamma_{\text{VEO}} = \pm 250\%$ and the actuator displacements were increased for each subsequent test. Unfortunately, specimens FCD-B2 and FCD-B3 could not be tested to failure dynamically, because the laboratory actuators did not have adequate displacement capacity at large velocities.

4.3.6.3.2 Quasi-Static Ultimate Tests

A clamping device was installed using built-up HSS sections and thick steel plates with holes for 8-1” A193 Grade 87 threaded rods that were pre-tensioned. This locked the movement of the FCD VE-steel layers together. Once the locking device was in place, the actuator was cycled back and forth with continuing displacement until failure of the built-up I-sections for FCD-A1 and RBSs for FCD-B1, FCD-B2 and FCD-B3. Figures 4.42 and 4.43 show the clamping devices for specimens FCD-A and FCD-B, respectively.

4.3.7 Full-Scale FCD Test Results

All of the harmonic characterization test results are located in Appendix D, the wind and earthquake time-history results in Appendix E and the ultimate static and dynamic test results in Appendix F.

4.3.7.1 FCD Force Calculations

The Free Body Diagram and rigid body kinematics of the test setup along with relevant geometry for both specimens are located in Figures 4.44 and 4.45.
The assumptions that were used to calculate the FCD shear force are:
1) Half the actuator force is transferred axially to the stiff channels, i.e. \( P_{\text{Channel}} = P_{\text{Act}} / 2 \) with the other half transferred to the right base pin,

2) There is no axial force in the VE material,

3) The moment at the center of the VE material is zero,

4) There is not a significant moment generated by the VE material in bending,

5) The stiff axial sections cause the walls to move to the same lateral displacement,

6) The friction forces are minimal,

7) The inertia forces in the walls are not significant

Using the first assumption that half the actuator force is transferred to the stiff axial sections, \( P_{\text{Channel}} = P_{\text{Act}} / 2 \), and taking the moment about the bottom right pin of the right wall with the FCD cut at the center of the VE material, the shear force is calculated as:

\[
F_{\text{FCD}} = \frac{P_{\text{Act}} h_{\text{Act}}}{L_{\text{CL}}}
\]  

(4.18)

where \( h_{\text{Act}} = 4087 \text{mm} \) is the actuator height, \( L_{\text{CLA}} = 5500 \text{mm} \) and \( L_{\text{CLB}} = 6000 \text{mm} \) are the lengths between base pins for specimen FCD-A and FCD-B respectively. The FCD shear force is calculated as \( F_{\text{FCD}A} = 0.743 P_{\text{Act}} \) and \( F_{\text{FCD}B} = 0.681 P_{\text{Act}} \), respectively.

All of these effects were more prevalent in the small amplitude tests compared to the larger amplitude tests. Overall, the test instrumentation confirmed that these assumptions were reasonable.

It was found that the linear uni-axial gauges on the flanges as well as the rosettes gauges on the webs of the FCD were not overly useful to obtain the FCD shear force, because they were placed in significantly disturbed regions and therefore typical beam equations used to calculate stress could not be utilized.

### 4.3.7.2 Local FCD Displacement Calculation

Figure 4.46 shows two photographs of the deformed shape of FCD-A3 under a lateral displacement applied by the two actuators. Figure 4.47 shows three photographs of a push cycle and a pull cycle deformed shape of FCD-B3. As can be seen in the photos the primary deformation occurs in the VE material in shear.

In addition to the VE material shear deformation, there were deformations in the steel elements cantilevered from the wall, as well as pull-out of the end-plate anchorage cast into the walls. To capture the relative deformation of each element in the FCD, local instrumentation was attached to stiff aluminum elements that were cantilevered off both the right and left walls, as shown in Figure 4.48.

The average FCD shear deformation, \( u_{\text{FCD}} \), was determined from the average of the change in length, \( \Delta \), of four of the string potentiometers cantilevered off the right and left walls as:

\[
u_{\text{FCD}} = \frac{[\Delta_{S_{1}} + \Delta_{S_{5}}] - (\Delta_{S_{1}} + \Delta_{S_{5}})}{2}
\]  

(4.19)
The average midspan FCD displacement at the center of the VE material relative to the left and right walls, \( u_{\text{MidL}} \) and \( u_{\text{MidR}} \), respectively, was determined from the change in length, \( \Delta \), of the three string potentiometers at the vertical steel plates, bonded to the VE material layers, relative to the right and left walls:
a) Push Cycle

b) Pull Cycle

Figure 4.47 FCD-B3: Deformed Shape

\[ u_{MidL} = \frac{(\Delta s_2 + \Delta s_3 + \Delta s_4)}{3} \]  \hspace{1cm} (4.20)

\[ u_{MidR} = \frac{(\Delta s_2 + \Delta s_3 + \Delta s_4)}{3} \]  \hspace{1cm} (4.21)
Although the steel plates and the end-plate connections are relatively rigid, the total FCD shear displacement is not equivalent to the total VE material shear displacement. This is caused by the contribution of three main sources at small amplitude levels:

1) Elastic bending deformations in the FCD built-up steel plates.
2) Elastic shear deformations in the FCD built-up steel plates.
3) End-plate rotations.

Determining the various contribution to the built-up assembly losses are important FCD design criteria. The deformed shape of the FCD as well as the individual deformation contributions to the total FCD deformations are located in Figures 4.49 and 4.50, respectively.

\[ u_{AL} = (u_{S_{FL}} - u_{MidL}) + \Delta_{SS_{F}} \]  
(4.22)

\[ u_{AR} = (u_{S_{FR}} - u_{MidR}) + \Delta_{SS_{R}} \]  
(4.23)

The average VE Material displacement, \( u_{VE} \), as illustrated in Figure 4.50a, can be expressed as the total assembly deformation subtracted from the total shear displacement as:

\[ u_{VE} = u_{FCD} - (u_{AR} + u_{AL}) \]  
(4.24)

Figure 4.49 FCD Exaggerated Deformed Shape
The FCD end-plate uplift was measured using another two small LVDTs glued to aluminum angles which were attached rigidly to the aluminum cantilever arms 100 mm from the wall connection hanging in front of the FCD end-plate. Small cracking amplitudes of less than 0.5 mm were observed in some of the larger amplitude tests, however there were no visual cracks observed in the concrete at the FCD end-plate, as seen in Figure 4.51.
The angle of rotation of the end-plate connection at low level displacement amplitudes can be calculated approximately as:

\[ \alpha_{EPR} \approx \frac{\Delta_{EPR}}{h_{LVDT}} \]  

\[ \alpha_{EPL} \approx \frac{\Delta_{EPL}}{h_{LVDT}} \]  

(4.25)  
(4.26)

Where \( h_{LVDT} \) is the distance of the LVDT to the rotation point of the end-plate; for low level displacements the rotation point is assumed at the middle of the end-plate. The total losses related to end-plate rotations, illustrated in Figure 4.50d, are estimated as:

\[ u_{EP} = \frac{L_{CS}}{2}(\alpha_{EPR} + \alpha_{EPL}) \]  

(4.27)

Where \( L_{CS} \) is the clearspan length between the wall segments.

For the lower amplitude tests for both dampers FCD-A and FCD-B (WSHC and HSHC) the end-plate rotation was calculated as approximately 10% of the losses and for larger amplitude tests (USHC) the end-plate rotation contributes about 15% of the losses, possibly related to small cracks at the concrete and end-plate connection, not seen visually.

The relative contributions to the shear and bending losses could be estimated theoretically using beam theory or using a finite element program.

### 4.3.7.3 FCD Property Calculations

The equations used to determine the mechanical properties of the FCD and VE material from the full-scale test data is similar to the procedure used in the small-scale test in Section 4.2.3, but adapted to take into account the elastic stiffness losses in the FCD itself as described in Section 4.3.4.

**STEP 1** The FCD shear deformation, \( u_{FCD} \), and the VE material displacements are calculated using Eqs. (4.19) and Eq. (4.24), respectively:

\[ u_{FCD} = \frac{1}{2}([\Delta_{S1F} + \Delta_{S5F}] - [\Delta_{S5R} + \Delta_{S1R}]) \]  

(4.28)

\[ u_{VE} = u_{FCD} - (u_{AR} + u_{AL}) \]  

(4.29)

**STEP 2** Each individual hysteretic cycle is counted when the FCD displacement crosses zero. The energy dissipated per cycle of the FCD, \( E_{FCD} \), and the VE material, \( E_{VED} \), are calculated as:

\[ E_{FCD} = \int F_{FCD} du_{FCD} \]  

(4.30)

\[ E_{VED} = \int F_{FCD} du_{VE} \]  

(4.31)
STEP 3) The viscous damping coefficients of the FCD and VE material, $c_{FCD}$ and $c_{VE}$, are determined using the average displacement amplitudes for each cycle, $u_{FCD0}$ and $u_{VE0}$, and the circular frequency $\omega_0 = 2\pi f_0$, respectively:

\[
c_{FCD} = \frac{E_{FCD}}{\pi \omega_0^2 u_{FCD0}^2} \tag{4.32}
\]
\[
c_{VE} = \frac{E_{VE}}{\pi \omega_0^2 u_{VE0}^2} \tag{4.33}
\]

STEP 4) The phase angle of the FCD and VE material, $\theta_{FCD}$ and $\theta_{VE}$, are calculated using the average force amplitudes for each cycle, $F_{FCD0}$ and $F_{VE0}$, respectively as:

\[
\theta_{FCD} = \arcsin \left( \frac{c_{FCD} \omega_0 u_{FCD0}}{F_{FCD0}} \right) \tag{4.34}
\]
\[
\theta_{VE} = \arcsin \left( \frac{c_{VE} \omega_0 u_{VE0}}{F_{VE0}} \right) \tag{4.35}
\]

STEP 5) The elastic stiffness coefficient of the FCD and VE material, $k_{FCD}$ and $k_{VE}$, respectively are then calculated as:

\[
k_{FCD} = \frac{F_{FCD0}}{u_{FCD0}} \cos \theta_{FCD} \tag{4.36}
\]
\[
k_{VE} = \frac{F_{VE0}}{u_{VE0}} \cos \theta_{VE} \tag{4.37}
\]

STEP 6) From the elastic stiffness coefficient of the VE material, $k_{VE}$, the shear storage modulus of the VE material, $G_E$, is obtained by multiplying the elastic stiffness coefficient by the height of the VE layers, $h$, divided by the total area of the VE layer undergoing shear, $A$:

\[
G_E = \frac{k_{VE} h}{A} \tag{4.38}
\]

STEP 7) The shear loss modulus, $G_c\omega$, is calculated by multiplying the viscous damping coefficient of the VE material, $c$, by the height of the VE layers, $h$, divided by the total area of the VE layer undergoing shear, $A$, multiplied by the circular frequency $\omega_0 = 2\pi f_0$:

\[
G_c\omega = \frac{ch}{A \omega_0} \tag{4.39}
\]

STEP 8) The loss factor of the VE material, $\eta$, is calculated as:

\[
\eta = \frac{G_c\omega}{G_E} \tag{4.40}
\]
4.3.7.4 Harmonic Characterization Tests

4.3.7.4.1 Working Harmonic Strain Characterization Tests

4.3.7.4.1.1 FCD-A

Working harmonic strain characterization tests were conducted for each of the specimens as the first subset of tests. The targeted VE material strain was \( \gamma_0 = 50\% \) at three frequencies \( f_0 = 0.1, 0.3, 0.5 \text{Hz} \) for 500 cycles. A summary of the average of the FCD-A test results from specimens FCD-A1, FCD-A2 and FCD-A3 are shown in Table 4-20 for the first and last cycle of the tests. The temperature measured was from the handheld thermometer before and after the tests. Graphs of the full force versus displacement curves for the frequencies \( f_0 = 0.1, 0.3, 0.5 \text{Hz} \) for FCD-A2 are located in Figure 4.52. The FCD stiffness and damping coefficients, \( k_{FCD} \) and \( c_{FCD} \), respectively versus the cycle number are located in Figures 4.53a and 4.53b and the shear storage modulus, \( G_E \), and the loss factor, \( \eta \), are located in Figures 4.54a and 4.54b, respectively.

<table>
<thead>
<tr>
<th>Test Name</th>
<th>( f_0 ) Hz</th>
<th>Cycle</th>
<th>( T_{VE} ) °C</th>
<th>FCD Properties</th>
<th>VE Material Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( u_{FCD0} ) mm</td>
<td>( F_0 ) kN</td>
</tr>
<tr>
<td>WSHC1</td>
<td>0.1</td>
<td>1</td>
<td>21.6</td>
<td>3.75</td>
<td>197.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>499</td>
<td>23.9</td>
<td>3.91</td>
<td>180.4</td>
</tr>
<tr>
<td>WSHC2</td>
<td>0.3</td>
<td>1</td>
<td>21.8</td>
<td>3.85</td>
<td>274.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>499</td>
<td>25.7</td>
<td>4.11</td>
<td>238.2</td>
</tr>
<tr>
<td>WSHC3</td>
<td>0.5</td>
<td>1</td>
<td>22.7</td>
<td>4.26</td>
<td>341.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>499</td>
<td>27.8</td>
<td>4.59</td>
<td>277.3</td>
</tr>
</tbody>
</table>
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

Figure 4.52 FCD-A2: Working Strain Harmonic Characterization

Figure 4.53 FCD-A2: WSHC FCD Properties

Figure 4.54 FCD-A2: WSHC VE Material Properties
4.3.7.4.1.2 FCD-B

The same set of tests were run for FCD-B specimens, the average results for the first and last cycles of the FCD-B2 and B-3 tests are given in Table 4-21. The full FCD and VE material hystereses are shown in Figure 4.55, followed by the damping and stiffness properties of the FCD in Figure 4.56 and the VE material properties over the duration of the test in Figure 4.57. Figure 4.58 displays the ThermaCAM test results obtained during the test. Figure 4.58b displays the temperature of 3 points for each test at the central VE layer, with point 1 at approximately the middle of the VE layer length, point 2 at approximately 1/3 of the VE layer length and point 3 at approximately 1/6 of the VE layer length for all three frequencies. Figure 4.58a displays the temperature across the damper cross section at lines 1, 2 and 3 at the beginning and end of the tests. The ThermaCAM images are located at the beginning, middle and end of the three WSHC tests in Figures 4.58c, 4.58d and 4.58e.

<table>
<thead>
<tr>
<th>Test Name</th>
<th>$f_0$ Hz</th>
<th>Cycle</th>
<th>$T_{FE}$ °C</th>
<th>$u_{FCD0}$ mm</th>
<th>$F_0$ kN</th>
<th>$k_{FCD}$ kN/mm</th>
<th>$c_{FCD}$ kN s/mm</th>
<th>VE Material Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>WSHC1</td>
<td>0.1</td>
<td>1</td>
<td>22.4</td>
<td>3.86</td>
<td>177.8</td>
<td>41.9</td>
<td>30.3</td>
<td>$u_{VE0}$ mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>499</td>
<td>24.7</td>
<td>3.97</td>
<td>169.2</td>
<td>38.9</td>
<td>28.0</td>
<td></td>
</tr>
<tr>
<td>WSHC2</td>
<td>0.3</td>
<td>1</td>
<td>24.3</td>
<td>4.26</td>
<td>230.5</td>
<td>50.5</td>
<td>10.28</td>
<td>2.06</td>
</tr>
<tr>
<td></td>
<td></td>
<td>499</td>
<td>27.5</td>
<td>4.44</td>
<td>213.2</td>
<td>44.5</td>
<td>9.59</td>
<td>2.37</td>
</tr>
<tr>
<td>WSHC3</td>
<td>0.5</td>
<td>1</td>
<td>23.5</td>
<td>4.57</td>
<td>280.2</td>
<td>58.0</td>
<td>6.33</td>
<td>1.78</td>
</tr>
<tr>
<td></td>
<td></td>
<td>499</td>
<td>27.5</td>
<td>4.77</td>
<td>255.6</td>
<td>50.0</td>
<td>6.07</td>
<td>2.32</td>
</tr>
</tbody>
</table>

**TABLE 4-21 Average of WSHC Tests of FCD-B2 and B-3**
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

Figure 4.55 FCD-B2: Working Strain Harmonic Characterization

Figure 4.56 FCD-B2: WSHC FCD Properties

Figure 4.57 FCD-B2: WSHC VE Material Properties

FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS
Figure 4.58 FCD-B2: WSHC ThermaCAM Temperature Data Summary
4.3.7.4.2 Higher Strain Harmonic Characterization Tests

4.3.7.4.2.1 FCD-A

Higher strain harmonic characterization tests were conducted for specimens FCD-A1 and FCD-A2. The targeted VE material strain was $\gamma_0 = 100\%$ at three frequencies $f_0 = 0.1, 0.2, 0.3 \text{Hz}$ for 100 cycles. A summary of the average of the FCD-A test results from specimens FCD-A1 and FCD-A2 are located in Table 4-22 for the first and last cycle of the tests. The temperature measured was from the handheld thermometer before and after the tests. Graphs of the full force versus displacement curves for the frequencies $f_0 = 0.1, 0.2, 0.3 \text{Hz}$ for FCD-A2 are located in Figure 4.59. The FCD stiffness and damping coefficients, $k_{FCD}$ and $c_{FCD}$, versus the number of cycles are located in Figures 4.60a and 4.60b, respectively, and the shear storage modulus, $G_E$, and the loss factor, $\eta$, are located in Figures 4.61a and 4.61b, respectively.

### TABLE 4-22 Average of HSHC Tests of FCD-A1 and A2

<table>
<thead>
<tr>
<th>Test Name</th>
<th>$f_0$ Hz</th>
<th>Cycle</th>
<th>$T_{VE}$ °C</th>
<th>$u_{FCD0}$ mm</th>
<th>$F_0$ kN</th>
<th>$k_{FCD}$ kN/mm</th>
<th>$c_{FCD}$ kNs/mm</th>
<th>$u_{VE0}$ mm</th>
<th>$G_E$ MPa</th>
<th>$G_{E\omega}$ MPa</th>
<th>$\eta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSHC1</td>
<td>0.1</td>
<td>1</td>
<td>22.6</td>
<td>7.74</td>
<td>373</td>
<td>43.5</td>
<td>33.1</td>
<td>4.60</td>
<td>0.126</td>
<td>0.082</td>
<td>0.649</td>
</tr>
<tr>
<td></td>
<td></td>
<td>99</td>
<td>24.7</td>
<td>7.96</td>
<td>330</td>
<td>37.0</td>
<td>29.8</td>
<td>5.58</td>
<td>0.102</td>
<td>0.066</td>
<td>0.646</td>
</tr>
<tr>
<td>HSHC2</td>
<td>0.2</td>
<td>1</td>
<td>22.8</td>
<td>7.88</td>
<td>459</td>
<td>52.7</td>
<td>20.0</td>
<td>4.99</td>
<td>0.161</td>
<td>0.121</td>
<td>0.751</td>
</tr>
<tr>
<td></td>
<td></td>
<td>99</td>
<td>25.6</td>
<td>8.17</td>
<td>395</td>
<td>43.3</td>
<td>17.34</td>
<td>5.71</td>
<td>0.125</td>
<td>0.087</td>
<td>0.701</td>
</tr>
<tr>
<td>HSHC3</td>
<td>0.3</td>
<td>1</td>
<td>22.8</td>
<td>8.00</td>
<td>513</td>
<td>58.6</td>
<td>13.91</td>
<td>4.60</td>
<td>0.189</td>
<td>0.151</td>
<td>0.799</td>
</tr>
<tr>
<td></td>
<td></td>
<td>99</td>
<td>25.5</td>
<td>8.37</td>
<td>435</td>
<td>46.7</td>
<td>12.22</td>
<td>5.58</td>
<td>0.139</td>
<td>0.101</td>
<td>0.732</td>
</tr>
</tbody>
</table>
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

Figure 4.59 FCD-A2: Higher Strain Harmonic Characterization

Figure 4.60 FCD-A2: HSHC FCD Properties

Figure 4.61 FCD-A2: HSHC VE Material Properties
4.3.7.4.2.2 FCD-B

The same set of HSHC tests were run for specimen FCD-B. The average results for the first and last cycles of the FCD-B2 and B-3 tests are given in Table 4-23. The full FCD and VE material hystereses are shown in Figure 4.62, followed by the damping and stiffness properties of the FCD in Figure 4.63 and VE material properties over the duration of the test in Figure 4.64. Figure 4.65 displays the ThermaCAM test results obtained during the test. Figure 4.65b displays the temperature of 3 points for each test at the central VE layer, with point 1 at approximately the middle of the VE layer length, point 2 at approximately 1/3 of the VE layer length and point 3 at approximately 1/6 of the VE layer length for all three frequencies. Figure 4.65a displays the temperature across the damper cross section at lines 1, 2 and 3 at the beginning and end of the tests. The ThermaCAM images are located at the beginning, middle and end of both WSHC tests in Figures 4.65c, 4.65d and 4.65e.

<table>
<thead>
<tr>
<th>Test Name</th>
<th>$f_0$ (Hz)</th>
<th>Cycle</th>
<th>$T_{VE}$ ($^\circ$C)</th>
<th>$u_{FCD0}$ (mm)</th>
<th>$F_0$ (kN)</th>
<th>$k_{FCD}$ (kN/mm)</th>
<th>$c_{FCD}$ (kNs/mm)</th>
<th>$u_{VE0}$ (mm)</th>
<th>$G_E$ (MPa)</th>
<th>$G_c$ ($G_c^{60}$)</th>
<th>$\eta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSHC1</td>
<td>0.1</td>
<td>1</td>
<td>24.1</td>
<td>7.83</td>
<td>335</td>
<td>39.3</td>
<td>26.7</td>
<td>5.02</td>
<td>0.120</td>
<td>0.084</td>
<td>0706</td>
</tr>
<tr>
<td>HSHC1</td>
<td>0.1</td>
<td>99</td>
<td>26.0</td>
<td>8.04</td>
<td>310</td>
<td>35.2</td>
<td>25.0</td>
<td>5.45</td>
<td>0.103</td>
<td>0.071</td>
<td>692</td>
</tr>
<tr>
<td>HSHC2</td>
<td>0.2</td>
<td>1</td>
<td>23.5</td>
<td>8.04</td>
<td>416</td>
<td>47.7</td>
<td>15.8</td>
<td>4.44</td>
<td>0.162</td>
<td>0.127</td>
<td>779</td>
</tr>
<tr>
<td>HSHC2</td>
<td>0.2</td>
<td>99</td>
<td>26.0</td>
<td>8.30</td>
<td>379</td>
<td>41.9</td>
<td>14.3</td>
<td>4.97</td>
<td>0.135</td>
<td>0.098</td>
<td>728</td>
</tr>
<tr>
<td>HSHC3</td>
<td>0.3</td>
<td>1</td>
<td>25.5</td>
<td>7.58</td>
<td>410</td>
<td>50.3</td>
<td>10.5</td>
<td>4.23</td>
<td>0.171</td>
<td>0.126</td>
<td>732</td>
</tr>
<tr>
<td>HSHC3</td>
<td>0.3</td>
<td>99</td>
<td>27.7</td>
<td>7.85</td>
<td>374</td>
<td>43.9</td>
<td>9.75</td>
<td>4.80</td>
<td>0.140</td>
<td>0.099</td>
<td>707</td>
</tr>
</tbody>
</table>
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

Figure 4.62 FCD-B3: Higher Strain Harmonic Characterization

Figure 4.63 FCD-B3: HSHC FCD Properties

Figure 4.64 FCD-B3: HSHC VE Material Properties
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

Figure 4.65 FCD-B3: HSHC ThermaCAM Temperature Data Summary
### 4.3.7.4.3 Ultimate Strain Harmonic Characterization Tests

#### 4.3.7.4.3.1 FCD-A

Ultimate strain harmonic characterization tests at $f_0 = 0.1, 0.2, 0.3\text{Hz}$ to targeted strains of $\gamma_0 = 200\%$ for 10 cycles are conducted on two samples FCD-A1 and FCD-A2. The average results of the first and final cycles are displayed in Table 4-24. The comparison between samples showed very similar results for all harmonic tests. Figure 4.66 shows the FCD and the VE material hysteresis of the USHC tests and Figures 4.67 and 4.68 display the FCD and the VE material properties versus cycle.

<table>
<thead>
<tr>
<th>Test Name</th>
<th>$f_0$ Hz</th>
<th>Cycle</th>
<th>$T_{VE}$ $^\circ\text{C}$</th>
<th>FCD Properties</th>
<th>VE Material Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$u_{FCD0}$ mm</td>
<td>$F_0$ kN</td>
</tr>
<tr>
<td>USHC1</td>
<td>0.1</td>
<td>1</td>
<td>22.0</td>
<td>14.96</td>
<td>674</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9</td>
<td>23.0</td>
<td>15.27</td>
<td>606</td>
</tr>
<tr>
<td>USHC2</td>
<td>0.2</td>
<td>1</td>
<td>22.1</td>
<td>14.39</td>
<td>758</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9</td>
<td>23.3</td>
<td>14.73</td>
<td>695</td>
</tr>
<tr>
<td>USHC3</td>
<td>0.3</td>
<td>1</td>
<td>21.7</td>
<td>15.06</td>
<td>871</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9</td>
<td>22.9</td>
<td>15.46</td>
<td>798</td>
</tr>
</tbody>
</table>
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

Figure 4.66 FCD-A2: Ultimate Strain Harmonic Characterization Tests

Figure 4.67 FCD-A2: USHC FCD Properties

Figure 4.68 FCD-A2: USHC VE Material Properties
4.3.7.4.3.2 FCD-B

Ultimate strain harmonic characterization tests at $f_0 = 0.1, 0.2, 0.3\text{Hz}$ to targeted strains of $\gamma_0 = 200\%$ for 10 cycles are conducted on sample FCD-B2. The average USHC test results for FCD-B are shown in Table 4-25. The hystereses, FCD properties, VE material properties and the ThermaCAM temperature data are located in Figures 4.69, 4.70, 4.71 and 4.72, respectively.

<table>
<thead>
<tr>
<th>Test Name</th>
<th>$f_0$</th>
<th>Cycle</th>
<th>$T_{VE}$</th>
<th>$u_{FCD0}$</th>
<th>$F_0$</th>
<th>$k_{FCD}$</th>
<th>$c_{FCD}$</th>
<th>$u_{VE0}$</th>
<th>$G_E$</th>
<th>$G_{C\omega}$</th>
<th>$\eta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>USHC1</td>
<td>0.1</td>
<td>1</td>
<td>23.0</td>
<td>16.80</td>
<td>662</td>
<td>35.8</td>
<td>26.1</td>
<td>11.00</td>
<td>0.115</td>
<td>0.084</td>
<td>0.733</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9</td>
<td>25.1</td>
<td>17.13</td>
<td>616</td>
<td>32.4</td>
<td>24.9</td>
<td>11.77</td>
<td>0.149</td>
<td>0.073</td>
<td>0.772</td>
</tr>
<tr>
<td>USHC2</td>
<td>0.2</td>
<td>1</td>
<td>22.4</td>
<td>15.98</td>
<td>766</td>
<td>44.4</td>
<td>14.46</td>
<td>8.83</td>
<td>0.149</td>
<td>0.122</td>
<td>0.817</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9</td>
<td>NA</td>
<td>16.35</td>
<td>719</td>
<td>40.3</td>
<td>13.96</td>
<td>9.73</td>
<td>0.124</td>
<td>0.103</td>
<td>0.831</td>
</tr>
<tr>
<td>USHC3</td>
<td>0.3</td>
<td>1</td>
<td>23.3</td>
<td>17.21</td>
<td>868</td>
<td>47.0</td>
<td>9.72</td>
<td>8.63</td>
<td>0.182</td>
<td>0.156</td>
<td>0.859</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9</td>
<td>24.4</td>
<td>17.70</td>
<td>811</td>
<td>42.3</td>
<td>9.38</td>
<td>9.78</td>
<td>0.147</td>
<td>0.126</td>
<td>0.861</td>
</tr>
</tbody>
</table>
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

Figure 4.69 FCD B3: Ultimate Strain Harmonic Characterization

Figure 4.70 FCD-B3: USHC FCD Properties

Figure 4.71 FCD-B3: USHC VE Material Properties
Figure 4.72 FCD-B3: USHC ThermaCAM Temperature Data Summary
4.3.7.4.4 Harmonic Characterization Tests Conclusions

The harmonic characterization tests determined the FCD modelling and design properties. The FCD and VE material properties were relatively stable for the duration of the tests as shown in the test results. There is an interaction between the stiffness of the built-up steel sections and the VE material in shear; i.e. there are elastic losses (end-plate rotations and the built-up steel bending and shear deformations) that reduce the amount of shear deformation occurring in the VE material and reduce the stiffness and damping of the FCD. The stiffness of the built-up steel sections is more important at the higher frequencies of loading, because of the relative stiffness of the VE material compared to that of the adjacent steel sections. The FCD properties reflect the dependency of the VE material with respect to frequency, ambient temperature, strain and internal temperature.

4.3.7.5 Representative FCD Time-Histories

4.3.7.5.1 Test Results

Figures 4.73 and 4.74 display summaries of the SLS and ULS across-wind time histories and the SLS and ULS along-wind time-histories of the 85-storey case study analytical results. Twelve short time-histories, 10 earthquakes and the 2 wind time-histories were applied to the specimens one after another for sample FCD-B3. The first set was targeting the FCD design strains discussed in Table 4.3.6.2. After the first test set at the design strains was conducted, a second set of tests at 1.5 times the design level strains was conducted and then finally a third set of tests at 2 times the design level strains for the case study building. Two times the targeted design level strains corresponded to $\gamma_{Max} = 300\%$ for the earthquake time histories, $\gamma_{Max} = 270\%$ for the across-wind time histories and $\gamma_{Max} = 340\%$ for the along-wind time-histories. Similar tests were performed for all other specimens. Figures 4.75, 4.76 and 4.77 display summaries of the twelve short time-histories. Note that each test was conducted with approximately 3 minutes of the time in-between tests and that was not enough time to cool the VE material. Figure 4.78 displays the ThermaCAM results recorded during the time-histories.
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

Figure 4.73 FCD-B3: 85-Storey Case Study Across-wind Results Summary
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

Figure 4.74 FCD-B3: 85-Storey Case Study Along-wind Results Summary

Along-wind SLS
a) Displacement History

Along-wind ULS
d) Displacement History

e) FCD Hysteresis

f) VE Material Hysteresis

ThermaCAM Temperature Data
g) Temperature History

h) Temperature Across FCD

i) Along-wind SLS

j) Along-wind ULS

Figure 4.74 FCD-B3: 85-Storey Case Study Along-wind Results Summary
Figure 4.75 FCD-B3: 85-Storey Case Study Time Histories, 2 Times Design Displacement (1/3)
Figure 4.76 FCD-B3: 85-Storey Case Study Time Histories, 2 Times Design Displacement (2/3)
### Displacement History

<table>
<thead>
<tr>
<th>Actuator</th>
<th>FCD</th>
<th>VEM</th>
</tr>
</thead>
</table>

#### a) Loma Prieta (2) 1989

![Graph](image1.png)

#### b) Kobe 1995

![Graph](image2.png)

#### c) Along-wind

![Graph](image3.png)

#### d) Across-wind

![Graph](image4.png)

### FCD Hysteresis

<table>
<thead>
<tr>
<th>Force (kN)</th>
<th>$F_{\text{Max}} = 1004 \text{ kN}$</th>
</tr>
</thead>
</table>

### VE Material Hysteresis

<table>
<thead>
<tr>
<th>Force (kN)</th>
<th>$F_{\text{Max}} = 1004 \text{ kN}$</th>
</tr>
</thead>
</table>

---

Figure 4.77 FCD-B3: 85-Storey Case Study Time Histories, 2 Times Design Displacement (3/3)
4.3.7.5.2 Conclusions from Time-History Tests

From the time-history tests the following conclusions were drawn:

1) The FCDs did not yield under the large displacement time-histories.
2) The VE material properties were stable and predictable for all design level wind time-histories and the temperature did not rise significantly during the tests.
3) The VE material properties were stable and predictable for two times the DLE loads and the temperature did not rise significantly during any of the tests.
4) The along-wind damper and VE material properties can be determined using the method of superposition, separating the mean and dynamic components.
5) Wind-induced vibrations are dominated by the first few modes of vibration and the VE and FCD response can be estimated using the VE material properties at those fundamental frequencies.
6) Earthquake-induced loading of the FCD involves higher mode participation.
4.3.7.6 Ultimate Dynamic Tests

4.3.7.6.1 FCD-A

FCD-A2 and FCD-A3 were tested dynamically to failure. The actuator displacement amplitude was increased until failure of the built-up steel I-sections. Figures 4.79a, 4.79b, 4.80a and 4.80b display both the FCD hystereses and VE material hystereses for all tests. The summary of the test results from FCD-A2 and FCD-A3 are shown in Tables 4-26 and 4-27, displaying the maximum FCD and VE displacements. There is a VE material deformation limit and an FCD force limit at large FCD displacements. As can be seen in the tests, as the actuator displacement and FCD shear displacement are increased, the response changes from a primarily VE material response with linear elastic behaviour in the steel sections, to a combination of VE material response and plastic response in the steel sections. When the steel sections yield, the response is characterized by a viscoelastic-plastic response, producing a “cat’s eye” hysteresis, as seen in Figures 4.79 and 4.80.

<table>
<thead>
<tr>
<th>TABLE 4-26 FCD-A2, Summary of Ultimate Dynamic Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Name</td>
</tr>
<tr>
<td>-----------</td>
</tr>
<tr>
<td>UD1</td>
</tr>
<tr>
<td>UD2</td>
</tr>
<tr>
<td>UD3</td>
</tr>
<tr>
<td>UD4</td>
</tr>
<tr>
<td>UD5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TABLE 4-27 FCD-A3, Summary of Ultimate Dynamic Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Name</td>
</tr>
<tr>
<td>-----------</td>
</tr>
<tr>
<td>UD1</td>
</tr>
<tr>
<td>UD2</td>
</tr>
<tr>
<td>UD3</td>
</tr>
<tr>
<td>UD4</td>
</tr>
<tr>
<td>UD5</td>
</tr>
<tr>
<td>UD6</td>
</tr>
</tbody>
</table>
Figure 4.79 FCD-A2: Summary of Ultimate Dynamic Tests at a Frequency of 0.2 Hz for 10 Cycles

Figure 4.80 FCD-A3: Summary of Ultimate Dynamic Tests at a Frequency of 0.3 Hz for 5 Cycles
The photographs of specimen FCD-A2, shown in Figure 4.81, were taken after yielding began in the built-up steel I-sections. White wash flaking began just in front of the toe of the flange weld, on both the top and bottom flanges. The yielding progressed through the flange and then almost completely through the web section between the bolted section and the end-plate. Fracture was instigated at the toe of the flange welds on the top and bottom flanges and then progressed through the thin flanges continuing through the thick web sections, as seen in Figure 4.82. Also as seen in the photo, there is no observed cracking or damage in the concrete walls at the end-plate connection. Figure 4.83 shows some close-up photos of the crack surfaces. Figure 4.84 shows the removal of the specimen, with all the damage concentrated in the built-up I-sections, with no tearing occurring in VE material, as can be seen in Figure 4.85.
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

Fork Configuration Dampers (FCDs) for Enhanced Dynamic Performance of High-Rise Buildings

Figure 4.83 FCD-A2: Fracture Surfaces

Figure 4.84 FCD-A2: Removal of Specimen after Tests

Figure 4.85 FCD-A2: No Damage Observed in VE Material after all Tests
4.3.7.6.2 FCD-B

Attempts were made to fail FCD-B2 and FCD-B3 dynamically by increasing the harmonic displacement; unfortunately the actuators did not have enough displacement capacity at large velocities. Figures 4.86 and 4.87 display both the FCD hysteresis and the VE material hysteresis for increasing actuator displacement amplitudes until the actuator capacity was reached for both specimens FCD-B2 and FCD-B3, respectively. The summary of the results from these tests are shown in Tables 4-28 and 4-29. As in the FCD-A tests, there is a VE material deformation limit and an FCD force limit at large FCD displacements. As the FCD displacement increases, the FCD response changes from primarily a VE material response with linear elastic behaviour in the steel sections, to a combination of VE material response and elasto-plastic steel section response, seen in Figures 4.86 and 4.87.

<table>
<thead>
<tr>
<th>Test Name</th>
<th>Number of Cycles, $N_{Cycles}$</th>
<th>Frequency, $f_0$ (Hz)</th>
<th>Max Force, $F_{Max}$ (kN)</th>
<th>Max Shear Deflection, $u_{FCDMax}$ (mm)</th>
<th>Max VE Displacement, $u_{VEMax}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>UD1</td>
<td>3</td>
<td>0.2</td>
<td>1218</td>
<td>48.9</td>
<td>23.5</td>
</tr>
<tr>
<td>UD2</td>
<td>3</td>
<td>0.2</td>
<td>1271</td>
<td>55.0</td>
<td>24.7</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Name</th>
<th>Number of Cycles, $N_{Cycles}$</th>
<th>Frequency, $f_0$ (Hz)</th>
<th>Max Force, $F_{Max}$ (kN)</th>
<th>Max Shear Deflection, $u_{FCDMax}$ (mm)</th>
<th>Max VE Displacement, $u_{VEMax}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>UD1</td>
<td>3</td>
<td>0.2</td>
<td>1008</td>
<td>27.7</td>
<td>16.3</td>
</tr>
<tr>
<td>UD2</td>
<td>3</td>
<td>0.2</td>
<td>1131</td>
<td>38.4</td>
<td>21.1</td>
</tr>
<tr>
<td>UD3</td>
<td>3</td>
<td>0.2</td>
<td>1236</td>
<td>48.7</td>
<td>26.1</td>
</tr>
<tr>
<td>UD4</td>
<td>3</td>
<td>0.2</td>
<td>1265</td>
<td>58.0</td>
<td>35.7</td>
</tr>
<tr>
<td>UD5</td>
<td>3</td>
<td>0.2</td>
<td>1315</td>
<td>65.6</td>
<td>31.8</td>
</tr>
<tr>
<td>UD6</td>
<td>3</td>
<td>0.2</td>
<td>1308</td>
<td>76.1</td>
<td>41.5</td>
</tr>
</tbody>
</table>

Figure 4.88 shows the white wash flaking that is evident at the location of the RBS in both the flange and the web of the RBS. Figure 4.89 shows a picture after all dynamic tests were completed, where only minor tears at the edge of the VE material can be seen.
CHAPTER 4: Experimental Validation of Fork Configuration Dampers (FCDs)

FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

Figure 4.86 FCD-B2: Summary of Ultimate Dynamic Tests at a Frequency of 0.2 Hz for 3 Cycles

\[ \text{Legend} \]
- UD1: \( F_{\text{max}} = 1205 \text{ kN}, T_{\text{res}} = 26.5^\circ C \)
- UD2: \( F_{\text{max}} = 1270 \text{ kN}, T_{\text{res}} = 25.0^\circ C \)

Figure 4.87 FCD-B3: Summary of Ultimate Dynamic Tests at a Frequency of 0.2 Hz for 3 Cycles

\[ \text{Legend} \]
- UD1: \( F_{\text{max}} = 1008 \text{ kN}, T_{\text{res}} = 25.6^\circ C \)
- UD2: \( F_{\text{max}} = 1131 \text{ kN}, T_{\text{res}} = 24.3^\circ C \)
- UD3: \( F_{\text{max}} = 1236 \text{ kN}, T_{\text{res}} = 25.7^\circ C \)
- UD4: \( F_{\text{max}} = 1264 \text{ kN}, T_{\text{res}} = 25.7^\circ C \)
- UD5: \( F_{\text{max}} = 1315 \text{ kN}, T_{\text{res}} = \text{NA} \)
- UD6: \( F_{\text{max}} = 1308 \text{ kN}, T_{\text{res}} = \text{NA} \)
Figure 4.88 FCD-B3: White Wash After Ultimate Dynamic Test 5 (UD5)

Figure 4.89 FCD-B3: VE Material after Ultimate Dynamic Tests

Minor Superficial Tearing at Layer Edges
4.3.7.6.3 Conclusions from Ultimate Dynamic Test

The yield and ultimate forces measured during the tests were higher than the predicted design basis forces. This is primarily due to the higher actual steel properties than were assumed in design. When both FCD-A and FCD-B specimens were tested dynamically to large amplitude displacements, the built-up steel sections began to yield, limiting deformations in the VE material and resulting in a viscoelastic-plastic FCD response, characterized by a “cat’s eye” hysteresis. There was minimal observed VE material damage or concrete-steel end-plate connection damage. FCD-B displayed a more ductile response than FCD-A in these ultimate dynamic tests. In the FCD-A tests it was possible to generate a full fracture of the built-up steel I-sections. In the FCD-B tests, the response was very ductile with the yielding in the RBS section protecting the flange welds. However because of the actuator limitations, the FCD-B specimens could not be tested dynamically to failure.

4.3.7.7 Ultimate Static Tests

4.3.7.7.1 FCD-A

Figure 4.90 displays the hysteresis obtained from FCD-A1 when the clamping device was installed. Figure 4.91 displays some photographs at the onset of the crack formation just beyond the toe of the weld at the built-up I-section-end-plate interface. Figure 4.92 shows the deformed shape of the clamped FCD.

![Figure 4.90 FCD-A1: Ultimate Static Test with Clamping Device](image-url)
Figure 4.91 FCD-A1: End-Plate Flange Connection Crack Formation

Figure 4.92 FCD-A1: Overall Deformed Shape with Clamping Device

a) Push Cycle   b) Pull Cycle   c) End of Tests   d) Right Top Flange

Figure 4.93 FCD-A1: End-plate Flange Connection at End of Tests
4.3.7.7.2 FCD-B

The hystereses of specimens FCD-B2 and FCD-B3 with the clamping device are shown in Figure 4.94. As can be seen by comparing Figure 4.91 and Figure 4.94 FCD-B had significantly more displacement capacity after yield than FCD-A. Figure 4.95 shows FCD-B2 prior to the clamped test. The white wash flaking indicates that yielding begins at the RBS flanges and progresses into the web sections. Figure 4.96 shows the push and pull deformed shape of the total setup with the FCD clamped. Subject to large displacements, the flanges buckle first in compression initiating lateral buckling of the adjacent web, while the opposite flanges are yielding in tension. After a number of cycles of the flange buckling and web lateral buckling, both the compression and tension flanges and webs begin to buckle in the same direction as can be seen in the deformed shape in Figure 4.97 and Figure 4.98. Eventually, after numerous back and forth cycles at large displacements, the out-of-plane buckling causes cracking in the tension flanges as can be seen in Figure 4.99.

![Figure 4.94 FCD-B: Ultimate Static Tests with Clamping Device](image)

![Figure 4.95 FCD-B2: Clamped FCD, Prior to Ultimate Static Tests](image)

Fork Configuration Dampers (FCDs) for Enhanced Dynamic Performance of High-Rise Buildings
Figure 4.96 FCD-B3: Ultimate Static Tests Deformed Shape

Figure 4.97 FCD-B2: Deformed Shape at Large Displacements
Figure 4.98 FCD-B2: Removal of Specimen After Tests

Figure 4.99 FCD-B2 Cracks at End of Test
4.3.7.7.3 Ultimate Static Test Conclusions

No significant damage occurred in the concrete-steel end-plate connection during the tests. FCD-B displayed a more ductile response than FCD-A. In the FCD-A tests, cracks began at the toe of the flange-end-plate welds and continued through the web sections. In the FCD-B tests, the response was very ductile, with the yielding occurring in the RBS directly, and eventually after many cycles of loading, buckling occurred in the web sections adjacent to the flange which then caused the entire web section to buckle out-of-plane. The ultimate strength was relatively the same for both static and dynamic tests.

4.3.8 FCD Design and Modelling Properties

The extensive testing of the small-scale and full-scale FCDs generated extensive data that can be used to derive the properties used for the design and modelling of high-rise buildings incorporating this device. A methodology has been developed to estimate the modeling properties for design applications using the equivalent spring-dashpot model described in Section 3.4.3. The ISD-111H VE material properties (NSEC 2006) have been validated through the UofT tests and EPM tests. The material properties for multiple strains and frequencies are given in Table 4-30:

<table>
<thead>
<tr>
<th>$f(Hz)$</th>
<th>$\gamma_0 = \pm 10%$</th>
<th>$\gamma_0 = \pm 50%$</th>
<th>$\gamma_0 = \pm 100%$</th>
<th>$\gamma_0 = \pm 200%$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T(°C)$</td>
<td>$G_E$ (MPa)</td>
<td>$f(Hz)$</td>
<td>$f(Hz)$</td>
<td>$f(Hz)$</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>0.129 0.196 0.327 0.435</td>
<td>0.126 0.194 0.327 0.443</td>
<td>0.123 0.191 0.327 0.446</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>0.083 0.114 0.180 0.229</td>
<td>0.083 0.115 0.176 0.229</td>
<td>0.078 0.109 0.172 0.227</td>
</tr>
<tr>
<td></td>
<td>$\eta$ (°C)</td>
<td>0.78 0.93 1.11 1.21</td>
<td>0.81 0.98 1.17 1.24</td>
<td>0.82 0.96 1.12 1.16</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>0.57 0.71 0.89 0.99</td>
<td>0.60 0.75 0.93 1.015</td>
<td>0.61 0.76 0.92 .98</td>
</tr>
</tbody>
</table>

Upper and lower design bound properties have been established for modelling the equivalent spring-dashpot model. Tables 4-31, 4-32 and 4-33 show the suggested across-wind, along-wind and earthquake design properties using the properties in Table 4-30. The material properties for a given frequency, temperature and strain are linearly interpolated between the tabulated values. The property values used in design are summarized below, but the full design strategy is described in detail with the properties in Chapter 6.

- **Wind upper bound properties** - Correspond to the beginning of a given loading at an ambient temperature of $T_0 = 20\,°C$ at the design strain level. The material properties in Tables 4-31 and 4-32 are from the VE material properties in Table 4-30, and the temperatures are based on a low room temperature. The material properties, $G_E$ and $\eta$, are deemed conservative for strength design of the FCD.
• **Wind lower bound properties** - correspond to the end of a given loading at the design strain level at the expected frequency. The material properties in Tables 4-31 and 4-32 are from the VE material properties in Table 4-30, and the temperatures are based on end temperatures observed in tests with similar loads and durations. The material properties, $G_E$ and $\eta$, are low for the design condition and deemed conservative for drift and adjacent member strength design.

• **Seismic properties** - The strategy for modelling VE material for earthquake design is different, because earthquake loads are significantly shorter and the VE material does not have time to heat up significantly. Fan (1998) suggests designing with an average temperature at the design level strain. Also, the building may respond in many modes of vibration. The properties suggested to be used for modelling and design are located in Table 4-33 at an average temperature of $T = 23^\circ C$ linearly interpolated from Table 4-30.

### TABLE 4-31 Across-wind Linearized Design ISD-111H Material Properties

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Design Bound</th>
<th>Design Strain</th>
<th>VE Temperature</th>
<th>$f(\text{Hz})$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\gamma_{\text{Max}}$ (%)</td>
<td>$T_{VE} (^\circ C)$</td>
</tr>
<tr>
<td>Across-wind SLS</td>
<td>Upper Bound</td>
<td>±100</td>
<td>20</td>
<td>0.123</td>
</tr>
<tr>
<td></td>
<td>Lower Bound</td>
<td>±100</td>
<td>25</td>
<td>0.101</td>
</tr>
<tr>
<td>Across-wind ULS</td>
<td>Upper Bound</td>
<td>±135</td>
<td>20</td>
<td>0.120</td>
</tr>
<tr>
<td></td>
<td>Lower Bound</td>
<td>±135</td>
<td>30</td>
<td>0.075</td>
</tr>
</tbody>
</table>

### TABLE 4-32 Along-wind Linearized Design ISD-111H Material Properties

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Design Bound</th>
<th>Design Strain</th>
<th>VE Temperature</th>
<th>$f(\text{Hz})$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\gamma_\text{S} \pm \gamma_{\text{Max}}$ (%)</td>
<td>$T (^\circ C)$</td>
</tr>
<tr>
<td>Along-wind SLS</td>
<td>Upper Bound</td>
<td>75±50</td>
<td>20</td>
<td>0.023</td>
</tr>
<tr>
<td></td>
<td>Lower Bound</td>
<td>75±50</td>
<td>25</td>
<td>0.018</td>
</tr>
<tr>
<td>Along-wind ULS</td>
<td>Upper Bound</td>
<td>100±70</td>
<td>20</td>
<td>0.020</td>
</tr>
<tr>
<td></td>
<td>Lower Bound</td>
<td>100±70</td>
<td>30</td>
<td>0.016</td>
</tr>
</tbody>
</table>

### TABLE 4-33 Earthquake Linearized Design ISD-111H Material Properties

<table>
<thead>
<tr>
<th>Design Strain</th>
<th>VE Temperature</th>
<th>$f(\text{Hz})$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_{\text{Max}}$ (%)</td>
<td>$T_{VE} (^\circ C)$</td>
<td>$G_E (MPa)$</td>
</tr>
<tr>
<td>±50</td>
<td>23</td>
<td>0.113</td>
</tr>
<tr>
<td>±100</td>
<td>23</td>
<td>0.110</td>
</tr>
<tr>
<td>±200</td>
<td>23</td>
<td>0.102</td>
</tr>
<tr>
<td>±400</td>
<td>23</td>
<td>0.087</td>
</tr>
</tbody>
</table>
4.3.8.1 Application to FCD

When modelling the FCD behaviour, the VE material properties cannot be transferred directly to the FCD stiffness and damping coefficients of the spring-dashpot model, because of the losses in the connecting steel assembly, as discussed in Section 3.4.3.

Using the equivalent VE technique the assembly stiffness, $k_A$, needs to be determined to model the relative stiffness of the VE material and the assembly. The assembly stiffness was determined from the tests from plotting the FCD shear force against the displacement losses and taking the slope. The assembly connection stiffnesses for FCD-A and B were determined to be $k_{AA} = 140 \text{kN/mm}$ and $k_{AB} = 110 \text{kN/mm}$, respectively. The $k_{FCD}$ and $c_{FCD}$ values are obtained using Eqs. (3.21) and (4.37).

As an example, the design properties of FCD-A for the across-wind loading design cases for a building with a natural frequency $f_0 = 0.2 \text{Hz}$ are calculated and displayed in Table 4-34.

### TABLE 4-34 FCD-A: Across-wind Design ISD-111H Material Properties

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Design Bound</th>
<th>VE Material Properties</th>
<th>FCD Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$G_E$ (MPa) $\eta$ $k_{VE}$ kN/mm $c_{VE}$ kNs/mm</td>
<td>$k_{FCD}$ kN/mm $c_{FCD}$ kNs/mm</td>
</tr>
<tr>
<td>Across-wind SLS</td>
<td>Upper Bound</td>
<td>0.161 0.885 72.3 50.9</td>
<td>55.3 20.3</td>
</tr>
<tr>
<td></td>
<td>Lower Bound</td>
<td>0.1255 0.79 56.3 35.4</td>
<td>45.0 23.5</td>
</tr>
<tr>
<td>Across-wind ULS</td>
<td>Upper Bound</td>
<td>0.160 0.895 71.8 51.1</td>
<td>55.3 20.3</td>
</tr>
<tr>
<td></td>
<td>Lower Bound</td>
<td>0.132 0.695 59.0 32.6</td>
<td>45.5 16.70</td>
</tr>
</tbody>
</table>

Using this procedure, the FCD modeling properties can be developed with the tabulated VE material properties and other FCD connections numerically.

4.3.8.2 Ultimate Strength Parameters

Based on the tests reported in this chapter, the ultimate FCD properties listed in Table 4-35 are suggested. The connection yield strengths are based on the design parameters, which were less than values observed in tests. The plastic strength is the average of the actual yield strength obtained from the coupon tests multiplied by the plastic section modulus. The ultimate strength maximum ultimate strength obtained during the tests of all three samples. The maximum VE material displacement, $u_{VE}$, is a strain of approximately 300%, which considers that the displacements during a very fast excitation could occur, causing large VE forces at smaller displacements. The maximum assembly displacement, $u_A$, and total FCD displacement, $u_{FCD}$, are conservative displacements observed in the tests at no more than 20% strength degradation. These design properties are proposed graphically in Figure 4.100.
### TABLE 4-35 Proposed Ultimate FCD Properties

<table>
<thead>
<tr>
<th></th>
<th>Factored Yield Strength</th>
<th>Nominal Yield Strength</th>
<th>Ultimate Strength</th>
<th>Maximum VE Material Displacement</th>
<th>Total Available Assembly Displacement</th>
<th>Total Available FCD Displacement</th>
<th>Total Available FCD Chord Rotation</th>
</tr>
</thead>
<tbody>
<tr>
<td>FCD-A</td>
<td>$\Phi F_y (kN)$</td>
<td>$F_y (kN)$</td>
<td>$F_u (kN)$</td>
<td>$u_{VE} (mm)$</td>
<td>$u_A (mm)$</td>
<td>$u_{FCD} (mm)$</td>
<td>$\theta_{FCD} (rads)$</td>
</tr>
<tr>
<td></td>
<td>607</td>
<td>675</td>
<td>1366</td>
<td>20</td>
<td>22</td>
<td>44</td>
<td>0.028</td>
</tr>
<tr>
<td>FCD-B</td>
<td>600</td>
<td>667</td>
<td>1331</td>
<td>20</td>
<td>100</td>
<td>120</td>
<td>0.057</td>
</tr>
</tbody>
</table>
4.3.9 Full-Scale Test Conclusions

The full-scale tests showed that the FCD as designed and tested can add distributed viscous damping when the VE material undergoes shear deformations, as seen in the harmonic characterization tests. The FCD and VE material properties are relatively stable for all design level time-histories, including full duration wind loads and even large amplitude earthquake loads. There is an interaction between the stiffness in series of the built-up steel sections and the VE material in shear; i.e., there are elastic losses (end-plate rotations and the built-up steel bending and shear deformations) that reduce the amount of shear deformation occurring in the VE material. The VE material properties are relatively low at static loads and show relaxation properties when held at a given displacement. During along-wind tests, where the damper is subjected to a combination of static and dynamic loads, the method of superposition can be used to obtain the total damper response.

The yield and ultimate tested forces are higher than the assumed design basis forces which may be due to higher steel properties, as seen in the coupon tests, and possibly due to different restraint conditions. When both FCD-A and FCD-B specimens were tested dynamically at large amplitude displacements the built-up steel sections began to yield limiting VE material deformation and FCD force, resulting in a viscoelastic-plastic FCD response or a “cat’s eye hysteresis”. Specimen FCD-B could not be tested to failure dynamically, due to limitations in the actuator displacement capacity at large velocities. No significant damage occurred in the VE material or in the concrete-steel end-plate connection. FCD-B displayed a more ductile response than FCD-A. In the FCD-A tests, cracks began at the toe of the flange welds which progressed through the web sections. In the FCD-B tests, the response was very ductile, the yielding began in the RBS section, and eventually after many cycles buckling occurred in the web sections adjacent to the flange which then caused both web sections to buckle out-of-plane. The FCD-A and FCD-B tests confirmed that the ultimate strength of the specimen was relatively the same for both static and dynamic tests.
CHAPTER 5: NUMERICAL MODELLING OF FORK CONFIGURATION DAMPERS (FCDs)

5.1 OVERVIEW OF DEVELOPMENT OF FCD MODELS

Three VE material models were considered in this chapter: i) the Kelvin-Voigt material model, ii) the generalized Maxwell model and iii) the generalized Maxwell model with a temperature correction. These VE material models were originally developed for axial brace configurations. To model the full shear behaviour of FCDs, the VE material models were then implemented in the shear direction in series with a spring, representing the stiffness of the built-up steel sections. The VE material models and the FCD models were validated against the VE material tests and the full-scale FCD tests. Conclusions on the modelling strategies and recommendations for practitioners are then presented.

5.2 VE MATERIAL MODELLING

Three models are used to represent the constitutive VE material behaviour: i) the Kelvin-Voigt material model (KVM), ii) the generalized Maxwell model with 4 Maxwell elements (GMM4) and iii) GMM4 with a temperature correction (GMM4TC). Figure 5.1 provides a summary of the VE material models that were considered.

![Figure 5.1 Summary of VE Material Models](attachment:image.png)
The KVM is a spring and dashpot modelled in parallel. This representation can capture the VE material behaviour only if the temperature, strain and frequency can be reasonably estimated and are not expected to vary substantially during the analysis. The KVM is the most commonly used model, because it is the simplest and easiest to implement within commercial software. This model is not effective if the temperature and frequency are not constant during an analysis. However, this model could be used with upper bound and lower bound temperatures to establish a design envelope for wind design, because the response is predominantly in a single mode of vibration.

The GMM4 (proposed by Fan 1998) uses multiple spring and dashpot models that account for variations in the temperature, strain and frequency. The index 4 refers to the number of Maxwell models in parallel (a Maxwell model is a spring and a dashpot in series) with a single Kelvin-Voigt model. The length of computational time increases with the number of Maxwell models. The use of four Maxwell models was suggested by Fan (1998), because it provides the best balance of accuracy and computational modelling time. The GMM4 is modified to include a temperature correction factor that takes into account convection and conduction and transfer of heat between the steel plates and VE material for long-duration loading. The GMM4 model with the temperature correction is labelled GMM4TC.

5.2.1 Kelvin-Voigt Model (KVM)

Hysteretic modelling and behaviour of VE materials and dampers subject to harmonic loading were discussed in Section 3.4.1 and are summarized in Figure 5.2. The Kelvin-Voigt solid behaviour under sinusoidal shear strains is illustrated in Figure 5.2a. The shear stress, \( \tau(t) \), can be expressed using the shear strain, \( \gamma(t) \), and shear strain rate, \( \dot{\gamma}(t) \), as (Zhang et al. 1989):

\[
\tau(t) = G_E \gamma(t) + G_C \dot{\gamma}(t)
\]

(5.1)

where \( G_E \) is the shear storage modulus and \( G_C \omega \) the shear loss modulus. Rather than the shear loss modulus, VE material manufacturers generally supply the loss factor, \( \eta \), for a measure of the energy dissipation, which is defined as:

\[
\eta = \frac{G_C \omega}{G_E}
\]

(5.2)

All \( G_E, G_C \) and \( \eta \) are functions of the frequency of excitation, the VE material temperature and the strain amplitude. The Kelvin-Voigt VE damper behaviour subject to sinusoidal shear displacements was derived in Chapter 3, and is illustrated in Figure 5.2b. Eq. (5.1) is transformed into a force-displacement relationship by accounting for the viscoelastic material shear thickness, \( h \), and shear area, \( A \):

\[
F(t) = ku(t) + c\dot{u}(t)
\]

(5.3)
Knowing the VE material properties, $G_E$, $G_C$, and $\eta$, at a given VE material temperature, strain and frequency gives the elastic stiffness and viscous damping coefficients.

### 5.2.2 Generalized Maxwell Model (GMM4)

The KVM model does not take into account property fluctuations caused by changes in temperature, strain or frequency over the duration of a wind storm or earthquake time-history. There are two commonly used constitutive models that can account for these property fluctuations: i) the fractional derivative model (Kasai et al. 1993) and ii) the generalized Maxwell model (Fan 1998). The fractional derivative model is more commonly used, however it relies on a formulation in the frequency domain, with real and imaginary functions, and the use of fractional derivatives, which are usually not well understood by structural engineers. The generalized Maxwell model (GMM4) utilizes multiple dashpot and spring
elements in series and parallel to form a model that can be implemented using techniques that are more common to structural engineers. For these reasons the GMM4 model was studied and implemented in this thesis. The models were developed for short-term loading only, such as earthquakes and therefore a correction factor was introduced in order to account for long-term loading such as wind vibrations.

5.2.2.1 Frequency, Temperature and Strain Dependent Modelling

The generalized Maxwell model is formed with one Kelvin-Voigt element (shown in Figure 5.3a) and \( n \) Maxwell elements connected in parallel (shown in Figure 5.3b). The generalized Maxwell model is shown in Figure 5.3c. In the generalized model there are a total of \( n + 1 \) parameters representing the spring elements and \( n + 1 \) parameters representing the dashpot elements, where \( n \) is the number of Maxwell elements (Fan 1998).

\[
\begin{align*}
G_E(\omega, T) &= G_0 + \sum_{m=1}^{n} \left\{ \frac{\left(\alpha_f(T, T_{ref})\omega\psi_{m,ref}\right)^2}{1 + \left(\alpha_f(T, T_{ref})\omega\psi_{m,ref}\right)^2} G_m \right\} \\
G_C(\omega, T)\omega &= \left(\alpha_f(T, T_{ref})\omega\beta_{0,ref}\right)G_0 + \sum_{m=1}^{n} \left\{ \frac{\left(\alpha_f(T, T_{ref})\omega\psi_{m,ref}\right)^2}{1 + \left(\alpha_f(T, T_{ref})\omega\psi_{m,ref}\right)^2} G_m \right\}
\end{align*}
\]

In the Maxwell model, the strain dependence of the shear storage modulus, \( G_E \), and the loss factor, \( \eta \), is accounted for by the self-heating of the VE material during loading (Lai et al. 1995). Therefore, \( G_E \) and \( \eta \) can be determined at smaller strain amplitudes (such as 10 or 50\%) and be applied to larger strain amplitudes (such as 200\%) by accounting for the self-heating of the VE material during loading (Fan 1998).

For the Maxwell model, the shear storage modulus, \( G_E \), the shear loss modulus, \( G_C \), and the loss factor, \( \eta \), are expressed as a function of the circular frequency, \( \omega \), and the VE temperature, \( T \):

\[
\begin{align*}
G_E(\omega, T) &= G_0 + \sum_{m=1}^{n} \left\{ \frac{\left(\alpha_f(T, T_{ref})\omega\psi_{m,ref}\right)^2}{1 + \left(\alpha_f(T, T_{ref})\omega\psi_{m,ref}\right)^2} G_m \right\} \\
G_C(\omega, T)\omega &= \left(\alpha_f(T, T_{ref})\omega\beta_{0,ref}\right)G_0 + \sum_{m=1}^{n} \left\{ \frac{\left(\alpha_f(T, T_{ref})\omega\psi_{m,ref}\right)^2}{1 + \left(\alpha_f(T, T_{ref})\omega\psi_{m,ref}\right)^2} G_m \right\}
\end{align*}
\]

Figure 5.3 Constitutive Models for VE Material Behaviour Modelling
CHAPTER 5: Numerical Modelling of Fork Configuration Dampers (FCDs)

\[ \eta(\omega, T) = \frac{G_c(\omega, T)}{G_E(\omega, T)} = \frac{(\alpha_f(T, T_{ref})G_0 + \sum_{m=1}^{n} \left[ \frac{(\alpha_f(T, T_{ref})\psi_{m, ref}^2}{1 + (\alpha_f(T, T_{ref})\omega\psi_{m, ref}^2)} \right] G_m)}{G_0 + \sum_{m=1}^{n} \left[ \frac{(\alpha_f(T, T_{ref})\omega\psi_{m, ref}^2}{1 + (\alpha_f(T, T_{ref})\omega\psi_{m, ref}^2)} \right] G_m} \]  

(5.8)

where \( \beta_{0, ref} \) and \( \psi_{m, ref} \) are the values of the modelling parameters, \( \beta_0 \) and \( \psi_m \), at the reference temperature \( T_{ref} \), and \( G_0 \) and \( G_m \) are generalized parameters of the model. A shifting function, \( \alpha_f(T, T_{ref}) \), is used to adjust the parameters from a reference temperature, \( T_{ref} \), to \( T \) using a temperature shifting function parameter, \( p \), as proposed by Kasai et al. (1993):

\[ \alpha_f(T, T_{ref}) = \left( \frac{T}{T_{ref}} \right)^p \]  

(5.9)

The generalized Maxwell model allows for an estimation of the increase in temperature, \( \Delta T \), caused by the VE material dissipating mechanical energy by transforming it to heat, described using the heat conduction equation:

\[ \kappa \nabla^2 T + \frac{dW}{dt} = \rho_s \frac{dT}{dt} \]  

(5.10)

where \( \kappa \) is the thermal conductivity of the VE material, \( T \) is the temperature, \( W \) is the dissipated energy per volume, \( \rho \) is the mass density of the VE material and \( s \) is the specific heat of the VE material. According to Fan (1998), \( \kappa \) is very small and can be neglected for short load durations, such as earthquakes, and therefore Eq. (5.10) can be rearranged and reduced to:

\[ \frac{dT}{dt} = \frac{1}{\rho_s} \frac{dW}{dt} \]  

(5.11)

The generalized Maxwell model allows the elastic work (reversible work done by the springs) to be distinguished from the dissipated energy (work done by the dashpots). The change in work done by the dashpots in Eq. (5.11) can be expressed as:

\[ dW = \left[ \frac{\tau_{kv0}^2}{\beta_0 G_0} + \sum_{m=1}^{n} \left( \frac{\tau_m^2}{\psi_m G_m} \right) \right] \]  

(5.12)

where \( \tau_{kv0} \) and \( \tau_m \) in Eq. (5.12), are the stresses in the Kelvin-Voigt dashpot and the maxwell dashpot for element \( m \). For the GMM4, model \( n \) is set to a value of 4, as recommended by Fan. Substituting Eq. (5.12) into Eq. (5.11) and rewriting in incremental form:

\[ \Delta T = \frac{\left[ \frac{\tau_{kv0}^2}{\beta_0 G_0} + \sum_{m=1}^{n} \left( \frac{\tau_m^2}{\psi_m G_m} \right) \right]}{\rho_s} \Delta t \]  

(5.13)
where $\tau_{kv}$ and $\tau_m$ are the stresses in the Kelvin-Voigt dashpot and the Maxwell dashpot for element $m$, respectively and $\Delta t$ is the time step.

This model assumes that VE material temperature and temperature rise is constant throughout the height and over the length and width for each layer of the VE material. Also, as stated previously, there is no transfer of heat between the VE material and the steel or the surrounding environment.

### 5.2.2.1.1 Generalized Maxwell Model (GMM4) Parameter Estimation

To use the generalized Maxwell model in an actual time history analysis the $2(n+1)$ parameters ($\beta_{0,ref}, G_0$ through $G_n$ and $\psi_1$ through $\psi_n$), plus the shifting function parameter, $\eta$, need to be defined. For the GMM4 model, which has $n = 4$ Maxwell models, the total number of parameters estimated is 10. Fan (1998) recommend using experimental data from tests at numerous temperatures and frequencies by performing two nonlinear least-squared regression analyses on the data. The proposed approach is:

**i) Estimate Parameters $G_0$ through $G_n$ and $\psi_1$ through $\psi_n$ and $\eta$**

- Perform a nonlinear least-square regression to minimize the error between Eq. (5.6), $G_E(\omega, T)$, and the test data, $G_{E_{exp}}(\omega, T)$, for various frequencies and temperatures, that is:

$$
\min \left( \sum_{i=1}^{x} \sum_{j=1}^{y} \left[ G_E(\omega_i, T_j) - G_{E_{exp}}(\omega_i, T_j) \right]^2 \right) \tag{5.14}
$$

where $x$ is a number of discrete frequencies and $y$ is the number of discrete temperatures. The parameters $G_0$ through $G_n$ and $\psi_1$ through $\psi_n$ and the shifting function parameter $\eta$ are estimated.

**ii) Estimate Parameter $\beta_{0,ref}$**

- Utilizing the parameters $G_0$ through $G_n$ and $\psi_1$ through $\psi_n$ and $\eta$ determined in the previous parameter estimation, perform a nonlinear least-square regression to minimize the error between Eq. (5.8), $\eta(\omega, T)$, and the test data, $\eta_{exp}(\omega, T)$, for various frequencies and temperatures, that is:

$$
\min \left( \sum_{i=1}^{x} \sum_{j=1}^{y} \left[ \eta(\omega_i, T_j) - \eta_{exp}(\omega_i, T_j) \right]^2 \right) \tag{5.15}
$$

where the $\beta_{0,ref}$ parameter is estimated, utilizing the parameters $G_0$ through $G_n$ and $\psi_1$ through $\psi_n$ from the previous step.
5.2.2.1.2 Newmark-Beta Time-History Implementation

The Newmark-Beta integration scheme is used to implement the general Maxwell model in a FE program. At each time step, $t_i$, the VE material temperature is updated and the shifting function $\alpha(T, T_{ref})$ is altered with the latest temperature. Figure 5.4a shows the spring and dashpot representation of the generalized Maxwell model. Note that in the Maxwell model there are internal degrees of freedom to obtain the relative displacements of each of the dashpots and springs.

**a) Spring and Dashpot Representation**

**b) Effective Condensed Model**

![Diagram of Maxwell model](image)

The equation of motion for the generalized Maxwell model at time, $t$, is:

$$[M]\{\ddot{u}(t)\} + [C]\{\dot{u}(t)\} + [K]\{u(t)\} = \{F(t)\}$$

(5.16)

assuming the mass is zero and rewriting Eq. (5.16) in incremental form at time step, $i$, or time, $t_i$:

$$[(C)\{\Delta\dot{u}\}_i] + [K]\{\Delta u\}_i = \{\Delta F\}_i$$

(5.17)

where the equations are:

$$[K] = \begin{bmatrix}
  k_1 & 0 & 0 & -k_1 \\
  0 & \ddots & \ddots & \ddots \\
  0 & 0 & k_m & -k_m \\
  -k_1 & -k_m & \sum_{m=1}^n k_m \\
\end{bmatrix}$$

(5.18)
\[
(C)_i = \begin{bmatrix}
(c_1)_i & 0 & 0 & 0 \\
0 & \cdots & 0 & \cdots \\
0 & 0 & (c_n)_i & 0 \\
0 & \cdots & 0 & (c_0)_i
\end{bmatrix}
\]

\[
\{(\Delta F)_i\} = \{F(t_i)\} - \{F(t_{i-1})\} = \begin{cases}
0 \\
0 \\
0 \\
(\Delta F_{VE})_i
\end{cases}
\]

\[
\{(\Delta u)_i\} = \{u(t_i)\} - \{u(t_{i-1})\} = \begin{cases}
(\Delta u_1)_i \\
\cdots \\
(\Delta u_n)_i \\
(\Delta u_{VE})_i
\end{cases}
\]

\[
\{(\Delta \dot{u})_i\} = \{\dot{u}(t_i)\} - \{\dot{u}(t_{i-1})\} = \begin{cases}
(\Delta \dot{u}_1)_i \\
\cdots \\
(\Delta \dot{u}_n)_i \\
(\Delta \dot{u}_{VE})_i
\end{cases}
\]

where the spring stiffness \(k_m\) and dashpot coefficients \((c_m)_i\) at time step \(i\) are expressed as:

\[
k_m = G_m A/h \quad \text{for} \quad m = 0, \ldots, n
\]

\[
(c_0)_i = A/h \alpha (T_r, T_{ref}) \beta_{0, ref} G_0
\]

\[
(c_m)_i = A/h \alpha (T_r, T_{ref}) \psi_{m, ref} G_m \quad \text{for} \quad m = 1, \ldots, n
\]

where \(A\) and \(h\) are the VE area and thickness. The incremental velocity, \(\{(\Delta \dot{u})_i\}\), expressed using the Newmark-Beta integration scheme:

\[
\{(\Delta \dot{u})_i\} = x_1 \{(\Delta u)_i\} + (-x_2 \{(\Delta \dot{u})_i\}) - x_3 \{(\ddot{u})_i\}
\]

where \(x_1\) through \(x_3\) are the Newmark-\(\beta\) constants:

\[
x_1 = \gamma/(\beta \Delta t)
\]

\[
x_2 = \gamma/\beta
\]

\[
x_3 = \Delta t(\gamma/(2\beta) - 1)
\]

where \(\gamma = 1/2\), \(\beta = 1/4\) and \(\gamma = 1/2\), \(\beta = 1/6\) are constants for the constant average acceleration and the linear acceleration integration schemes, respectively. The velocity, \(\{(\dot{u})_i\}\), and acceleration, \(\{(\ddot{u})_i\}\), in Eq. (5.26) at the previous step, step \(i-1\), can be written as:
\[
\begin{align*}
\{(\hat{u}_i)\} &= \{(\hat{u}_{i-1})\} + \{(\Delta \hat{u}_{i-1})\} = \{(\hat{u}_{i-1})\} + x_1\{(\Delta u)_{i-1}\} - x_2\{(\hat{u})_{i-1}\} - x_3\{(\hat{u})_{i-1}\} \\
\{(\hat{u})_i\} &= \{(\hat{u})_{i-1}\} + \{(\Delta \hat{u})_{i-1}\} = \{(\hat{u}_{i-1})\} + x_4\{(\Delta u)_{i-1}\} - x_5\{(\hat{u})_{i-1}\} - x_6\{(\hat{u})_{i-1}\}
\end{align*}
\] (5.30)

\[
\begin{align*}
\{(\hat{u})_i\} &= \{(\hat{u})_{i-1}\} + \{(\Delta \hat{u})_{i-1}\} = \{(\hat{u}_{i-1})\} + x_4\{(\Delta u)_{i-1}\} - x_5\{(\hat{u})_{i-1}\} - x_6\{(\hat{u})_{i-1}\}
\end{align*}
\] (5.31)

Substituting Eq. (5.26) into the incremental equation of motion, given by Eq. (5.17), the incremental effective static equilibrium equation can be obtained:

\[
\{(\hat{K}_i)\}\{(\Delta u)_i\} = \{(\Delta \hat{F})_i\}
\] (5.32)

where \([\hat{K}_i]\) is the effective stiffness matrix:

\[
\begin{align*}
[\hat{K}_i] &= [\hat{K}] + x_1[(C)_i] =
\begin{bmatrix}
  k_1 + x_1c_1 & 0 & 0 & -k_1 \\
  0 & \cdots & 0 & 0 \\
  0 & 0 & k_n + x_1c_n & -k_n \\
  -k_1 & \cdots & -k_n & k_0 + x_1c_0 + \sum_{m=1}^{n} k_m
\end{bmatrix}
\end{align*}
\] (5.33)

where \([(\Delta \hat{F})_i]\) is the effective load vector:

\[
\begin{align*}
\{(\Delta \hat{F})_i\} &= \{(\Delta F)_i\} + x_2[C]\{(\hat{u})_i\} + x_3[C]\{(\hat{u})_i\} =
\begin{bmatrix}
  x_2(c_1)(\hat{u}_1)_i + x_3(c_1)(\hat{u}_1)_i \\
  \cdots \\
  x_2(c_n)(\hat{u}_n)_i + x_3(c_n)(\hat{u}_n)_i \\
  (\Delta F_{VE})_i + x_2(c_0)(\hat{u}_0)_i + x_3(c_0)(\hat{u}_0)_i
\end{bmatrix}
\end{align*}
\] (5.34)

\[
\begin{align*}
\{(\Delta u)_i\} &= \{u(t_i)\} - \{u(t_{i-1})\} =
\begin{bmatrix}
  (u_1)_i \\
  \cdots \\
  (u_n)_i \\
  \{u_{VE}\}_i
\end{bmatrix}
\end{align*}
\] (5.35)

Using matrix manipulation and condensing out the \(n\) Maxwell element degrees of freedom, an expression for the condensed effective static equilibrium of \(u_{VE}\) in Figure 5.4 at time step \(i\) can be obtained:

\[
(\hat{k}_{VE}^*)_i(\Delta u_{VE})_i = (\hat{F}_{VE}^*)_i
\] (5.36)

where \((\hat{k}_{VE}^*)_i\) is the condensed effective stiffness, \((\Delta \hat{F}_{VE}^*)_i\) is the incremental condensed effective force at time step \(i\) and \((\Delta u_{VE})_i\) is the incremental damper displacement. The condensed effective stiffness and incremental condensed effective force at \(i\) are expressed as:

\[
(\hat{k}_{VE}^*)_i = k_0 + x_1(c_0)_i + \sum_{m=1}^{n} \left( k_m - \frac{k_m^2}{k_m + x_1(c_m)_i} \right)
\] (5.37)

\[
(\Delta \hat{F}_{VE}^*)_i = (\Delta F_{VE})_i + x_2(c_0)(\hat{u}_{VE})_i + x_3(c_0)(\hat{u}_{VE})_i + \sum_{m=1}^{n} \left( \frac{k_m}{k_m + x_1(c_m)_i} \right) \left[ x_2(c_m)(\hat{u}_m)_i + x_3(\hat{u}_m)_i \right]
\] (5.38)
To account for the self-heating in the analysis, at the end of each time step $i$, the stress is calculated for only the dashpots in the model and using Eq. (5.13) the change in temperature is obtained. This is then used as the new temperature at time step $i + 1$.

### 5.2.3 VE Parameter Estimation

#### 5.2.3.1 KVM Modelling Parameters

The VE material properties used for the KVM are located in Table 5-1.

<table>
<thead>
<tr>
<th>$T(°C)$</th>
<th>$f (Hz)$</th>
<th>$f (Hz)$</th>
<th>$f (Hz)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_0 = \pm50%$</td>
<td>0.1 0.3 1 2</td>
<td>0.1 0.3 1 2</td>
<td>0.1 0.3 1 2</td>
</tr>
<tr>
<td>$\gamma_0 = \pm100%$</td>
<td>0.126 0.194 0.327 0.443</td>
<td>0.123 0.191 0.327 0.446</td>
<td>0.116 0.183 0.316 0.411</td>
</tr>
<tr>
<td>$\gamma_0 = \pm200%$</td>
<td>0.083 0.115 0.176 0.229</td>
<td>0.078 0.109 0.172 0.227</td>
<td>0.069 0.101 0.165 0.224</td>
</tr>
</tbody>
</table>

The temperature is critical in the calibration of the KVM modelling parameters. The KVM does not have the ability to capture the self-heating of the VE material, but if the temperature is known, the VE material properties can be estimated. To assess the sensitivity of the KVM to temperature changes, a temperature was imposed on the model, assumed to vary linearly from the initial thermometer temperature to the final thermometer data. The hysteretic material properties, $G_E(f, \gamma, T)$ and $\eta(f, \gamma, T)$ are estimated from the VE material properties, for each time step, by interpolating the values in Table 5-1 for the cyclic frequency, $f$, the linearly varied temperature of the VE material at the strain amplitude, $\gamma_0$. This linear interpolation is just used to provide a comparison of the models, but is not a recommended analysis strategy.

#### 5.2.3.2 GMM4 Modelling Parameters

The VE material properties used for the GMM4 are given in Table 5-3. The only variable that needs to be identified before running the model is the ambient temperature of the VE material.

The modelling parameters of the GMM4 model are based on the VE properties, $G_E(f, T)$ and $\eta(f, T)$, obtained using the two nonlinear least-squared regression analyses over all the frequencies and temperatures as described in Section 5.2.2.1.1. The reference temperature used for the analysis is $T_{ref} = 24°C$. All the parameters that were obtained using the regression analysis are listed in Table 5-3.
Looking at the parameters obtained, the $G_3$ term is much larger and the term $\psi_{3,ref}$ is much smaller than the other Maxwell modelling parameters. At first glance it would seem that the $G_3$ coefficient would govern the response, however in the implementation of the generalized Maxwell model, the multiplication of the two parameters $\psi_{3,ref}G_3$ represents the damping coefficient, which is comparable to the other damping coefficients. Another interesting result for the parameter estimation is that the $\beta_{0,ref}$ term is negative. This does not make sense physically as it represents the damping coefficient of a dashpot, and is likely a function of the nonlinear fit and the VE properties in Table 5-2.

### 5.2.4 Comparison of the KVM and GMM4 with Small-Scale Test Results

The results of the KVM and GMM4 described above for harmonic characterization tests of the small scale VE material tests at frequencies of $f = 0.2Hz$ and three strain levels of $\gamma_0 = 100\%$, $200\%$ and $400\%$, are shown in Figures 5.5, 5.6 and 5.7, respectively. The input displacement used in the models is the recorded displacement from the two external potentiometers shown in Figure 4.3. In each of the three figures the experimental hysteresis results are shown for the first 60 seconds in Figures a and for the first
900 seconds in Figures e. They can be compared with the KVM predictions in Figures c and g and the GMM4 predictions in Figures d and h. The starting temperatures obtained using the handheld thermometer measurements for the models are shown in Figures b and f.

As can be seen in Figures c and g, the KVM gives reasonable results for the first cycle, but does not capture the losses in VE material properties due to self-heating. This is due to the fact that the model's VE properties, \( G_E \) and \( \eta \), are based on the linear approximation of the surface temperature of the VE material at the beginning and the end of the test. Therefore the KVM will not capture the temperature increase related to the self-heating during the first 60 seconds, particularly for larger strains.

A limitation of the tests was that the temperature was not controlled and the temperature measurements were from a handheld thermometer that only measured the surface temperature. The internal temperature was likely higher than the measurements made using the thermometer.

For the GMM4, as seen in Figures d (particularly in Figures 5.6d and 5.7d), the VE material self-heating results in an immediate drop in stiffness of the VE material properties and therefore better captures the temperature time-history relative to the KVM. However as the loading duration increases, the GMM4 overestimates the temperature increase that was measured experimentally. This is related to the fact that the heat conduction of the steel plates, which is neglected in the GMM4, tends to distribute the heat to the surrounding steel plates. This becomes more significant as the load duration increases, as was observed by Fan (1998). This effect can be seen in Figures h, where the analysis results are presented for 900 seconds of loading. The test was conducted for \( t = 7200 \) s and the measured temperature at the end of the test is plotted in orange, in Figure f, to emphasize that at \( t = 900 \) s the GMM4 significantly overestimates the temperature and therefore the VE material properties, \( G_E \) and \( \eta \), are underestimated. This is not important for earthquake loadings which are relatively short in duration, but this becomes significant for long-term loads, such as wind storms. From the long-term harmonic characterization tests in Section 4.2 it was shown that mechanical properties eventually reach a steady state level. A factor to account for this phenomenon using the GMM4 is developed in the following section to improve the modelling of the VE dampers for long duration loading.
CHAPTER 5: Numerical Modelling of Fork Configuration Dampers (FCDs)

207

FORK CONFIGURATION DAMPERS (FCDS) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

Figure 5.5 VE Material Modelling, UT-VEM5-3: at a Frequency of 0.2 Hz and Strain of 100%

60 seconds
a) Experiment

\[ F_{sw} = 12.1 \text{kN} \]

b) Temperature History

\[ F_{sw} = 13.9 \text{kN} \]

900 seconds
e) Experiment

\[ F_{sw} = 12.1 \text{kN} \]

f) Temperature History

\[ F_{sw} = 13.9 \text{kN} \]

g) KVM

\[ F_{sw} = 14.5 \text{kN} \]

h) GMM4

\[ F_{sw} = 13.9 \text{kN} \]

Fork Configuration Dampers (FCDs) for Enhanced Dynamic Performance of High-Rise Buildings
Figure 5.6 VE Material Modelling, UT-VEM5-3: at a Frequency of 0.2 Hz and Strain of 200%
5.2.4.1 Long-Term Loading Correction Factor

Some research has been conducted on long-term loading on VE dampers in Japan by Kasai et al. (1997) and Sato et al. (2005). Kasai et al. (1997) used the fractional derivative model discussed earlier and integrated the fractional derivatives over the height of the VE material layer to obtain the heat generated by the VE damper. Unfortunately, the full formulation of the heat conduction and heat convection relationships between the VE material and steel plates were not published in English. Sato et al. (2005) used a 3-D FE model with multiple solid elements over the thickness of the VE and steel layers along the width and height of a damper to model the conduction and convection of heat. Both approaches were able to capture the results adequately, but the validations were limited to very small-scale VE dampers.

Since the FCD was developed for both wind and earthquake vibrations, the long-term modelling effects are important and therefore a simple expression to describe the long-term loading was established to be included in the models.

The correction factor is based on a non-dimensional quantity, $\Lambda$, which is a function of: i) the
specific heat, \( s \), of the VE material, ii) the density of steel, \( \rho \), iii) the heat conduction factor, \( \kappa \), iv) the thickness of steel layers bonding the VE material layers, \( h_{\text{Steel}} \), v) the number of steel layers, \( N_{\text{Steel}} \), vi) the number of VE layers, \( N_{\text{VE}} \), and vii) the height of the VE material, \( h \), and the area of VE material, \( A \), and a \( \beta \) factor, which is a calibration factor fit to the experimental data results. \( \Lambda \) is expressed as:

\[
\Lambda = \frac{\beta s \rho AN_{\text{Steel}} h_{\text{Steel}}}{\kappa N_{\text{VE}} h}
\]

Eq. (5.13) is now modified by the correction factor for long-term loading, \( \Delta T_{\text{corr}} \):

\[
\Delta T_{\text{corr}} = \left( 1 - \frac{1}{e^{\Lambda/t}} \right) \frac{1}{\rho s} \left[ \frac{\tau_{k_0}}{\beta_0 G_0} + \sum_{m=1}^{n} \left( \frac{\tau_m}{\psi_m G_m} \right) \right] \Delta t
\]

The \( \beta \) factor was determined by calibrating it over multiple tests with frequencies from \( f = 0.1 \) to \( 0.5 \text{Hz} \) and strains from \( \gamma = 50 \) to \( 200\% \), comparing the temperature rise and \( G_E \) and \( \eta \) properties to the experimental properties. As the \( \beta \) factor is made smaller, the VE properties reach steady-state values more quickly and if it is made larger the steady-state values are reached more slowly. A \( \beta \) value of 0.005 was determined to give the best representation of the self-heating over the entire range of test data, however there was a large variation in the values of \( \beta \) for which the models matched the experimental data well. The variation in \( \beta \) was likely due to the fact that it covered such a large range of test cases. It may be more reasonable to use multiple \( \beta \) factors for the different test cases, grouping tests with similar strains or frequencies together. Also, within the range of about \( \beta = 0.001 \) to \( 0.010 \) the values result in relatively the same response for the test data considered. They do not cause large changes in the VE material properties or peak forces, they only cause the VE properties to approach steady-state more quickly or slowly. This modification was able to provide a limit to the temperature increase, however the factor may not be applicable to all situations and therefore it is recommended to use test data to determine a reasonable estimate of \( \Lambda \). More study is required to address the temperature correlation, it is also possible that another simple factor could achieve good results.

### 5.2.4.2 GMM4TC Long-Term Results

An example of an application of the temperature correction factor used in the GMM4TC with a \( \beta \) value of 0.009 for the test at a frequency of \( f = 0.2 \text{Hz} \) and strain of \( \gamma = 200\% \) for a longer duration of 7200 seconds is studied. The GMM4TC is compared to the KVM and the experimental results, shown in Figure 5.8. Again the experimental test results are shown in Figure 5.8a, the temperature history in Figure b and the hysteretic results in Figures c and d for the KVM and GMM4TC, respectively. The rate of temperature increase for GMM4TC, as seen in Figure b, decreases as the test progresses and begins to approach a steady state value towards the end of the test. Figure d shows that the GMM4TC exhibits a
change in the stiffness and damping properties similar to the experimental hysteretic results over the duration of the experiment. As can be seen using the temperature correction factor, the temperature does not increase as substantially as it did in Figure 5.6. Also, using this \( \beta \) value it was actually able to achieve the final measured temperature.

The overall predicted response from the KVM displayed in Figure c does not reflect the fact that the temperature changes much more rapidly at the start of the loading compared to the end. Unless the temperature is known over the full test duration, the KVM stiffness and damping properties can not be determined precisely. However, they could provide convenient upper bound and lower bound design properties if the temperature can be estimated at the beginning and the end of the time-history. For example, the maximum VE force in each of these tests would occur in the first cycle, because there is no reduction in stiffness or damping related to self-heating. The minimum force would occur towards the end of the time-history when the self-heating is at its maximum. These design bounds are the basis for the design properties that were suggested in Chapter 4. The design bounds will be explored further when modelling wind loads using the FCD.

---

**Figure 5.8 VE Material Modelling, UT-VEM5-3: at a Frequency of 0.2 Hz and Strain of 200%**
5.3 MODELLING TECHNIQUES FOR THE FCD RESPONSE

The VE material models discussed prior are now implemented in series with a spring model in order to model the full FCD behaviour in shear. There are three modelling techniques used to model the shear behaviour of the FCDs. They are validated by comparing the response of the models to the experimental results. The FCD forces from the experiment and the method to determine the FCD shear and VE shear deformations were described earlier in Chapter 4.

The experimental FCD input displacement signal, $u_{FCD}$, defines the displacement loading that is imposed on the models that are presented herein and the models calculate the FCD force and the VE material displacement $u_{VE} = u_{FCD} - u_A$. The shear stiffness of the assembly is calculated from the experimental results. For FCD-A, the stiffness is approximated as $k_{AA} = 140 \text{kN/mm}$ and for FCD-B, the stiffness is approximated as $k_{AB} = 110 \text{kN/mm}$.

The modelling based on these localized measurements was viewed as more useful than modelling the entire test set-up, because the FCD properties were the most important items obtained from the tests. Also, the stiffness and damping coefficients are required to model the FCDs in building structures. Also, the global test behaviour would have to include modelling the pin slackness that would complicate the model significantly without any additional information gained on modelling FCDs in real structures.

The models that had been previously developed for brace damper applications are transformed to represent the FCD behaviour in shear. To model a brace configuration VE damper, the formulation places a spring element in series with the VE material models. The spring element represents the stiffness of the brace and brace connection. When modelling the FCD behaviour, the formulation assumes all action is concentrated at the midspan of the FCD spanning between two rigid offset elements cantilevered from the walls. The rigid cantilevers are attached to the VE material on one side and are anchored into the concrete walls on the other. An effective stiffness of the built-up steel assemblies is estimated by approximating the assembly stiffness, $k_A$, from the test results, theoretical approximations or results from FE models. A spring representing the assembly stiffness is then used in series with the previously developed VE models. The formulation is described in detail below. The models that were developed are summarized in Figure 5.9 and described in order of complexity: i) the equivalent VE model (model 1, Figure 5.9b), ii) the spring KVM (model 2, Figure 5.9c) and iii) the spring GMM4TC (model 3, Figure 5.9d). The VE material is modelled using the KVM for models 1 and 2 and the GMM4TC for model 3.
5.3.1 Equivalent VE Model (Model 1)

Model 1 was originally developed for VE dampers in brace configurations with different brace stiffnesses (Kasai 2006) as described in Chapter 3. The formulation is adapted to represent the deformed shape of the FCD in shear as shown in Figure 5.10a. The method for a brace configuration uses an equivalent KVM (shown in Figure 5.10b) as a simplification of a KVM (presented in Section 5.2.1) in series with a spring. The model adapted for an FCD consists of a spring model in series with a KVM, rotated 90 degrees, to act in shear, at the center of the clearspan as shown in Figure 5.10b. Two rigid offsets meeting at the center span are connected by the equivalent VE model in shear to form model 1. The equivalent FCD damper stiffness, $k_{FCD}$, may be expressed as a combination of the assembly stiffness, $k_{A}$, and VE material elements as:

$$k_{FCD} = \frac{k_{A} \Gamma_{VE} k_{VE}}{k_{A} + \Gamma_{VE} k_{VE}} \quad (5.41)$$

where $\Gamma_{VE}$ is a factor, which is a function of the VE material stiffness, $k_{VE}$, and steel assembly stiffness, $k_{A}$, and the loss factor of the VE material, $\eta_{VE}$, is calculated as:

$$\Gamma_{VE} = 1 + \frac{\eta_{VE}^2}{1 + k_{A}/k_{VE}} \quad (5.42)$$

The equivalent FCD loss factor of the damper, $\eta_{FCD}$, is expressed as:

$$\eta_{FCD} = \frac{\eta_{VE}}{1 + (1 + \eta_{VE}^2)k_{VE}/k_{A}} \quad (5.43)$$

The equivalent damping coefficient is calculated as:

$$c_{FCD} = \frac{\eta_{FCD} k_{FCD}}{\omega_0} \quad (5.44)$$

![Figure 5.9 Summary of FCD Models](image)
Using this formulation the FCD behaviour in shear can be modelled using the effective shear properties with a single frame element acting in shear, which would be available in commercially available structural engineering programs. Note that the coefficients in the above formulation are only relevant if the frequency, temperature and strain can be estimated and the response is predominantly a single mode of vibration.

5.3.2 Spring Kelvin-Voigt Model (Model 2)

The second model used is the spring KVM (model 2) which consists of a linear elastic spring in series with the KVM (presented in Section 5.2.1) and modelled using two degrees of freedom (shown in Figure 5.11b). Model 2 is only a incremental jump from the KVM, however it allows for a simple extension to model viscoelastic plastic behaviour if the spring is modelled with an elasto-plastic member.

\[
\begin{align*}
\{C\} \{\Delta \dot{u}_i\} + \{K\} \{\Delta u_i\} &= \{(\Delta F)_i\} \\
\end{align*}
\]

where the equations are:
[\begin{align*}
[K] &= \begin{bmatrix}
k_{VE} + k_A - k_A \\
k_A & -k_A
\end{bmatrix} \\
[C] &= \begin{bmatrix}
c_{VE} & 0 \\
0 & 0
\end{bmatrix}
\end{align*}

(5.46) (5.47)

\{(\Delta F)\}_i = \{F(t_i)\} - \{F(t_{i-1})\} = \begin{bmatrix}
0 \\
(\Delta F_{FCD})_i
\end{bmatrix}

(5.48)

\{(\Delta u)\}_i = \{u(t_i)\} - \{u(t_{i-1})\} = \begin{bmatrix}
(\Delta u_{VE})_i \\
(\Delta u_{FCD})_i
\end{bmatrix}

(5.49)

\{(\dot{\Delta u})\}_i = \{\dot{u}(t_i)\} - \{\dot{u}(t_{i-1})\} = \begin{bmatrix}
(\dot{\Delta u}_{VE})_i \\
(\dot{\Delta u}_{FCD})_i
\end{bmatrix}

(5.50)

where the VE stiffness coefficient, \(k_{VE}\), and damping coefficient, \(c_{VE}\) are expressed as:

\[k_{VE} = G_E A/h\]

(5.51)

\[c_{VE} = G_C A/h\]

(5.52)

where \(A\) and \(h\) are the VE area and thickness, respectively. The incremental velocity, \(\{(\dot{\Delta u})\}_i\), expressed using the Newmark-\(\beta\) integration scheme as:

\[\{(\dot{\Delta u})\}_i = x_1\{(\Delta u)\}_i - x_2\{(\dot{u})\}_i - x_3\{(\ddot{u})\}_i\]

(5.53)

where \(x_1\) through \(x_3\) are the Newmark-\(\beta\) constants defined in Eqs. (5.27), (5.28) and (5.29). The velocity, \(\{(\dot{u})\}_i\), and acceleration, \(\{(\ddot{u})\}_i\), in Eq. (5.53) at the previous step, \(i-1\), can be written as:

\[\{(\ddot{u})\}_i = \{(\ddot{u})\}_{i-1} + \{(\Delta \dot{u})\}_{i-1} = \{(\ddot{u})\}_{i-1} + x_1\{(\Delta u)\}_{i-1} - x_2\{(\dot{u})\}_{i-1} - x_3\{(\ddot{u})\}_{i-1}\]

(5.54)

\[\{(\ddot{u})\}_i = \{(\ddot{u})\}_{i-1} + \{(\Delta \ddot{u})\}_{i-1} = \{(\ddot{u})\}_{i-1} + x_4\{(\Delta u)\}_{i-1} - x_5\{(\dot{u})\}_{i-1} - x_6\{(\ddot{u})\}_{i-1}\]

(5.55)

Substituting Eq. (5.53) into the incremental equation of motion, Eq. (5.45), the incremental effective static equilibrium equation can be obtained:

\[\{(\ddot{\Delta F})\}_i = \{(\Delta F)\}_i\]

(5.56)

where \([(\ddot{\Delta F})\] is the effective stiffness matrix. Using matrix manipulation and condensing out the VE degree of freedom, an expression for the condensed effective static equilibrium at time step \(i\) is obtained:

\[(\ddot{k}_{FCD}^*)_i\{(\Delta u)_{FCD}\}_i = \{(\ddot{\Delta F})_{FCD}^*)_i\]

(5.57)
CHAPTER 5: Numerical Modelling of Fork Configuration Dampers (FCDs)

where \( (\hat{k}_{FCD})_i \) is the condensed effective FCD stiffness, \( (\Delta F_{FCD}^*)_i \) is the incremental condensed effective FCD force at \( i \) and \( (\Delta u_{FCD})_i \) is the incremental FCD displacement. The effective condensed spring FCD is shown in Figure 5.11c. The condensed effective FCD stiffness and incremental condensed effective FCD force at \( i \) are expressed as:

\[
(\hat{k}_{FCD}^*)_i = k_A - \frac{k_A^2}{k_{VE} + x_1 c_{VE} + k_M}
\]

\[
(\Delta F_{FCD}^*)_i = (\Delta F_{FCD})_i + \frac{k_A}{k_{VE} + x_1 c_{VE} + k_M} \{ x_2 c_{VE}(\dot{u}_{FCD}) + x_3 c_{VE}u_{FCD} \}
\]

Similarly as in Section 5.3.1, the spring KVM shown in Figure 5.11b, can be transformed into the condensed effective FCD model shown in Figure 5.11c. The condensed effective stiffness, \( (\hat{k}_{FCD})_i \), and incremental effective FCD force, \( (\Delta F_{FCD}^*)_i \), at \( i \) are expressed as:

\[
(\hat{k}_{FCD}^*)_i = k_A - \frac{k_A^2}{k_{VE} + x_1 c_{VE} + k_M}
\]

\[
(\Delta F_{FCD}^*)_i = (\Delta F_{FCD})_i + \frac{k_A}{k_{VE} + x_1 c_{VE} + k_M} \{ x_2 c_{VE}(\dot{u}_{FCD}) + x_3 c_{VE}u_{FCD} \}
\]

Note that the coefficients for model 2 are only relevant if the frequency, temperature and strain can be reasonably estimated and the response is characterized by a single mode of vibration.

5.3.3 Generalized Maxwell Model (Model 3)

Model 3 in Figure 5.12b, is a spring in series with the GMM4TC. The GMM4TC presented in Section 5.2.2 captures the frequency, strain and temperature dependency of the VE material behaviour. The Newmark-Beta integration scheme is again used to implement the spring in series with the GMM4TC.

a) VE FCD Deformed Shape  b) Spring GMM4TC  c) Effective Condensed Spring GMM4TC

Figure 5.12 Modelling VE FCD with Generalized Maxwell Model (Model 3)
The incremental effective static equilibrium equation can be obtained for the spring in series with the GMM4TC as:

\[
[(\hat{K})_i]((\Delta u)_i) = \{\Delta \hat{F}\}_i
\]  

(5.62)

where the equations are:

\[
[(\hat{K})_i] = \begin{bmatrix}
(\hat{k}_{VE})_i + k_A - k_A & -k_A \\
-k_A & k_A \\
\end{bmatrix}
\]  

(5.63)

\[
\{\{\Delta F\}_i\} = \{F(t_i)\} - \{F(t_{i-1})\} = \begin{bmatrix} 0 \\ (\Delta F_{FCD})_i \end{bmatrix}
\]  

(5.64)

\[
\{\{\Delta u\}_i\} = \{u(t_i)\} - \{u(t_{i-1})\} = \begin{bmatrix} (\Delta u_{VE})_i \\ (\Delta u_{FCD})_i \end{bmatrix}
\]  

(5.65)

\[
\{\{\Delta \dot{u}\}_i\} = \{\dot{u}(t_i)\} - \{\dot{u}(t_{i-1})\} = \begin{bmatrix} (\Delta \dot{u}_{VE})_i \\ (\Delta \dot{u}_{FCD})_i \end{bmatrix}
\]  

(5.66)

where \((\hat{k}_{VE})_i\) is the condensed effective VE stiffness at time step \(i\), calculated in Eq. (5.37):

\[
(\hat{k}_{VE})_i = k_0 + x_1(c_0) + \sum_{m=1}^{n} \left( k_m - \frac{k_m^2}{k_m + x_1(c_m)} \right)
\]  

(5.67)

and using force equilibrium:

\[
(\Delta F_{FCD})_i = (\Delta F_A)_i = (\Delta F_{VE})_i
\]  

(5.68)

using matrix manipulation and condensing out the internal VE degree of freedom, an expression for the condensed effective static equilibrium of the damper at time step \(i\) is obtained:

\[
(\hat{k}_{FCD})_i(\Delta u_{FCD})_i = (\Delta \hat{F}_{FCD})_i
\]  

(5.69)

where \((\hat{k}_{FCD})_i\) is the condensed effective stiffness, \((\Delta \hat{F}_{FCD})_i\) is the incremental condensed effective force at \(i\) and \((\Delta u_{FCD})_i\) is the incremental FCD displacement. Figure 5.12c shows the effective condensed model 3. The condensed effective stiffness at \(i\) is expressed as:

\[
(\hat{k}_{FCD})_i = k_A - \frac{k_A^2}{(\hat{k}_{VE})_i + k_A}
\]  

(5.70)

or
the incremental condensed effective force is expressed as:

\[
(\hat{k}_{FCD}^*)_i = k_d - \frac{k_d^2}{k_0 x_1(c_0)_i + \sum_{m=1}^{n} \left( \frac{k_m^2}{k_m + x_1(c_m)_i} \right) + k_d}
\] (5.71)

To account for the self-heating in the time-history at the end of each time step \(i\) the stress is calculated for each dashpot in the VE generalized Maxwell model and using Eq. (5.13) or Eq. (5.40) the change in temperature is obtained. This is then used as the new temperature at time step \(i + 1\).

5.3.4 FCD Modelling Results

The numerical models outlined in the previous paragraphs are validated using the harmonic characterization, wind and earthquake test results. The results in the following sections are based on the formulation of the models using first principles and the parameters used in the models have not been fitted to match the hysteretic behaviour, they are based on the manufacturer material tests as used for the KVM and GMM4TC VE material validations in Section 5.2. Further optimization of the parameters could be made if a more accurate representation is required. The only items used from the tests are the estimates of the assembly stiffness determined in Section 4.3.8. Theoretical cantilever stiffness estimates could be used alternatively.

The temperature has shown to be a critical parameter for establishing correct VE material properties. Model 3 only requires the initial temperature measurement and estimates analytically the temperature at each time step as describe in the previous sections. ThermaCAM temperature data was captured for the FCD-B tests. To estimate the VE material properties for the FCD-B tests accurately, an average of the three measurements on the VE material are made using the ThermaCAM temperature data and used to define the material properties at each time step for the models 1 and models 2.

For the FCD-A tests modelled with model 1 and model 2, the temperature of the VE material is linearly interpolated from the recorded handheld temperature measured at the beginning and end of the tests. This will cause a slight overestimation of the VE material properties used by the models, because they are based on the surface VE temperature measurements which have been shown to underestimate the actual temperature.
Note the temperature used with models 1 and 2 are simply to show the temperature dependency of the material and a change in VE material properties over the duration of the tests. They also require the knowledge of the end temperature in order to define the linear temperature variation that is used in the model.

5.3.4.1 Velocity Sensitivity of Models 1 and 2

The force-displacement response for models 1 and 2 is sensitive to the velocity signal. Reasonable displacement profiles applied by actuators can give poor velocity signals due to precision of the testing equipment, as observed by researchers in Japan (Kasai et al., 2003 and Ooki et al., 2004). Using a noisy velocity signal with models 1 and 2 will cause an unrealistic damper response, because models 1 and 2 are calibrated for a specific harmonic signal and velocity profile. This is illustrated in Figure 5.13.

Figures 5.13a and 5.13b show the shear displacement and the shear velocity time-history signal for FCD-A2 subject to a frequency of $f = 0.5\text{Hz}$ and a targeted strain of $\gamma = 50\%$. Near the peaks of the velocity portion of the signal, the signal is very noisy due to instrument errors, testing equipment precision and pin slackness, even though the shear displacement is relatively harmonic. Figures 5.13c and 5.13e show the force generated by Eq. (5.3) with constant FCD shear stiffness and damping coefficients. As can be observed, the model 1 force response is highly dependent on the noisy velocity signal, which when compared to the experimental response in Figure 5.13d, is shown to be unrealistic. Model 2 shows a similar response as model 1, however is slightly less sensitive to the velocity signal. Model 3 is very good at avoiding these velocity sensitivity issues.

In order to validate the model for analysis, the shear displacement signal of the FCD was filtered to remove some of the noise. Figure 5.13f shows the filtered force-displacement hysteresis. As can be seen the main harmonic backbone is comparable to the experimental response. This smoothing of the displacement input signal was applied to all the harmonic characterization tests and wind time-histories, because the response is predominantly in only one mode of vibration. The earthquake displacement signal is not filtered, because it is difficult to separate out higher mode effects, which are real, from artificial signal noise. The response to earthquakes shows the extreme velocity dependence of models 1 and 2.

5.3.4.2 Harmonic Characterization Results

5.3.4.2.1 FCD-A

Results from the WSHC3 experiment ($f_0 = 0.5\text{Hz}$, targeted strain $\gamma_0 = 50\%$ for 500 cycles), HSHC3 experiment ($f_0 = 0.3\text{Hz}$, targeted strain $\gamma_0 = 100\%$ for 100 cycles) and the USHC1 experiment ($f_0 = 0.3\text{Hz}$, targeted strain $\gamma_0 = 100\%$ for 100 cycles) are shown in Figures 5.14, 5.15 and 5.16, respectively. The VE material and FCD experimental response obtained from the measurement shown in Figures a. Figures b, c and d show the modelling results of model 1, model 2 and model 3, respectively.
Both the VE material response and the overall FCD shear response are plotted for the experimental, model 2 and 3 results. Model 1 (equivalent VE model) only has the overall FCD shear response since it is a SDOF model.

The modelling results show that model 1 and model 2 have very similar results both in the maximum force and in overall hysteretic shape. This is mainly due to the fact that the properties used in these analyses are based on the same estimated temperature. The maximum force of models 1 and 2 is slightly larger than the experiment, related to the fact the VE material temperature is underestimated using the handheld thermometer. As seen in Figures d, the maximum force obtained using model 3, is less than what was obtained in models 1 and 2. This is likely due to the modelling parameters for model 3, that were obtained for a large range of frequencies \( f = 0.1 \) to \( 2 \text{Hz} \) and VE temperatures \( T = 10 \) to \( 40\text{°C} \). These tests were conducted in the low frequency range, \( f = 0.1 \) to \( 0.5\text{Hz} \) usually at an ambient temperature, so the model may not be as efficient for this type of tests.

Figures e show the built-up assembly force-displacement history. As can be seen from the experimental results the assembly force versus displacement plot of the experiment is nearly linear with an approximate value of about \( k_A = 140kN/mm \). The minor hysteresis observed in the assembly are not related to yielding or energy dissipation in the assembly. They are likely related to errors in measurement calculation or possibly due to pin slack.
Figures f shows the VE temperature response calculated using model 3 as well as the handheld VE thermometer measurements. It can be seen that the temperature obtained in model 3 is higher than the handheld temperature linear measurement approximation for almost all load levels. It does appear that model 3 may underestimate the self-heating, because it seems to approach a steady-state quicker than the experiment for the long duration tests.

Figures g and h show the FCD storage stiffness and damping coefficient over time. It can be seen that all models match the experimental FCD storage stiffness and damping coefficients reasonably well. Model 3 seems to follow the overall reduction of stiffness and damping with respect to the test duration better than models 1 and 2.
Figure 5.14 Modelling FCD-A2: WSHC3 at a Frequency of 0.5 Hz and a Targeted Strain of 50% for 500 Cycles
CHAPTER 5: Numerical Modelling of Fork Configuration Dampers (FCDs)

Figure 5.15 Modelling FCD-A2: HSHC3 at a Frequency of 0.3 Hz and a Targeted Strain of 100% for 100 Cycles
Figure 5.16 Modelling FCD-A2: USHC1 at a Frequency of 0.1 Hz and a Targeted Strain of 50% for 10 Cycles
5.3.4.2.2 FCD-B

The same set of harmonic characterization tests used to validate the models for FCD-A are used for FCD-B. The test results are shown in Figures 5.17, 5.18 and 5.19, respectively. The ThermaCAM temperature data was used with this set of characterization tests to obtain more accurate measurements of the VE material temperature. Models 1 and 2, now use the temperature data as an average of the three points on the VE material. This average ThermaCAM temperature is then used at each time step as the new VE temperature to calculate the new VE material properties instead of the linear variation used in the FCD-A tests. This updates the VE properties for more refined VE material properties for models 1 and 2.

Looking at the new set of figures, the VE material and FCD experimental response obtained from the tests is shown in Figures a. Figures b shows the temperature response of the three points in the ThermaCAM and the average of those points is used at each time step for models 1 and 2. The model 3 temperature-history results are shown again as well as the handheld VE thermometer measurements. As shown in Figures b, at the end of the tests, the ThermaCAM temperatures are higher for all characterization tests towards the end of the tests than the modelled temperature. The results show that the model 3 temperature data is more accurate for the shorter duration tests, but as the length of the tests increases the ThermaCAM temperature is generally higher. These results could be improved with a more refined long-duration correction factor.

Figures c, d and e show the modelling results for models 1, 2 and 3, respectively. The results show that models 1 and 2 have very similar results both in the maximum force, but also in the overall hysteretic shape. The maximum forces in models 1 and 2 are now predicted better since the temperature used to define the VE material properties at each time step have been further refined by the ThermaCAM measurements. As seen in Figures e, the model 3 force is less than observed in models 1 and 2. This is related related to the larger self-heating effects in the first few cycles for this model when compared to the ThermaCAM measurements. The handheld thermometer surface temperature measurements are lower than the VE temperature measured using the thermaCAM at the end of the tests. This supports the belief that the handheld VE measurements are actually lower than the actual temperature at the end of the tests. Since \( k_B \) is less than \( k_A \), the FCD-B hysteresis is more pinched than the FCD-A hysteresis and the displacement in the VE material is less than observed in FCD-A.

Figures f show the built-up steel assembly’s force-displacement history. As can be seen from the experimental tests, the assembly force versus displacement shows, nearly a linear response, with an approximate value of about \( k_A = 110kN/mm \). Again the slight hysteresis observed in the assembly is not related to yielding or energy dissipation in the assembly, it is likely related to the overall setup response and the FCD force calculation.
Figures g and h show the FCD storage stiffness and damping coefficient over time. It can be seen that all models appear to result in reasonable approximations of these properties. As observed in the hystereses, model 3 shows lower stiffness and damping at the beginning of the tests, but the damping and stiffness flatline and stabilize towards the end of the time-history. This is related to the temperature correction factor reaching a steady state value. Models 1 and 2 show decreases in the stiffness and damping that are directly related to the ThermaCAM results used to estimate the damping and stiffness properties.

Lastly the ThermaCAM images taken at the beginning, middle and end of the tests, are shown in Figures i. As observed in the ThermaCAM results, the VE material varies in temperature over the width and thickness of the VE material. The thermaCAM temperature data gives no information about the temperature through the height of the VE material, which may be more at mid-height than at the surface where it was measured.
Figure 5.17 Modelling FCD-B2: WSHC6 at a Frequency of 0.5 Hz and a Targeted Strain of 50% for 500 Cycles
Figure 5.18 Modelling FCD-B3: HSHC3 and a Frequency of 0.3 Hz at a Targeted Strain of 100% for 100 Cycles
CHAPTER 5: Numerical Modelling of Fork Configuration Dampers (FCDs)

FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

Figure 5.19 Modelling FCD-B3: USHC1 at a Frequency of 0.1 Hz and a Targeted Strain of 200% for 10 Cycles
5.3.4.3 Time-History Results

5.3.4.3.1 Wind Time-Histories

The results from the time-histories are investigated for the FCD-B sample. Figure 5.20 shows the ULS across-wind test results for the case study building which had a natural period of \( T_1 = 7.25 \, s \). Figure 5.20a shows the shear displacement input signal used for all the modelling and the FCD tests. Figure 5.20b shows the experimental hysteresis and Figure 5.20c shows the model 3 hysteresis for the full wind time-histories. The force is slightly lower than what was observed in the experiments. This is likely due to the modelling parameters used to define model 3, which were based on a large range of frequencies \((f = 0.1 \text{ to } 2 \, \text{Hz})\) and VE temperatures \((T = 10 \text{ to } 40 \, ^\circ\text{C})\). The displacement response was mainly concentrated in the natural frequency of vibration of the structure. Further refinements of the model parameters could be achieved by calculating the parameters for the VE material data around the frequency and temperature of interest.

To demonstrate the use of upper and lower bounds for design, three plots are presented for the time-histories of models 1 and 2 in Figures 5.20d and e, respectively. The first graph uses the ThermaCAM temperature data and therefore matches the FCD time-history fairly well, both in shape and maximum force. The second and third graphs show the response using the lower bound and upper bound design values at the structure’s frequency.

Figure 5.20f displays the temperature history over the duration of the test. For the ULS across-wind loading, the recommended lower bound \( T_{VELB} = 30 \, ^\circ\text{C} \) and upper bound \( T_{VEUB} = 20 \, ^\circ\text{C} \) are shown. These results are based on the suggested properties in Table 4-31. As shown, the temperature-history measured using the handheld thermometer and the ThermaCAM are well within the upper and lower bounds. As can be seen in this figure, the design bounds are slightly conservative, and therefore it may be possible to decrease the lower bound temperature, \( T_{VELB} = 30 \, ^\circ\text{C} \), since the temperature at the end of the study only reached about \( T_{VEEnd} = 25.5 \, ^\circ\text{C} \). Model 3 again underestimates the temperature increase due to self-heating near the end of the tests.

Finally the ThermaCAM images are located in Figure 5.20g. As can be seen in the figure at the end of the test, the temperature is well distributed over the thickness of the damper and the steel plate temperature is approximately the same as the VE material.
Figure 5.20 Modelling FCD-B3: AWTH ULS Time-History of the 85-Storey Case Study
5.3.4.3.2 Earthquake Time-Histories

Modelling results from two large displacement earthquake time-history tests conducted on FCD-B are shown in Figures 5.21 and 5.22. The earthquake records used for this comparison were Landers 1993 and Loma Prieta 1989. The results are for a building with a natural period of $T_1 = 7.25\, s$, although it is clear there are higher modes contributing to the measured results.

Figures a show the FCD shear displacement time-history. As can be seen for the Landers earthquake, the displacement-history response is mainly in the first mode of vibration, whereas the response for the Loma Prieta earthquake contains more higher mode contributions. Figures b show the experimental response and Figures c show the assembly force-displacement response. Compared to the wind and harmonic tests, the stiffness of the assembly is less. This could be due to the fact that the displacements are much larger and there may be small amounts of cracking in the concrete-end-plate connection. Also, it is possible, because of the large load levels that the steel elements reach the onset of yielding. The assembly stiffness at these locations is approximated as $k_A = 80\, kN/mm$.

Figures d, e and f show the hysteretic response of models 1, 2 and 3. The generalized Maxwell model (model 3) best matches the experimental results. This is related to the fact that the model can take into account the frequency, strain and temperature dependent properties of the VE material.

Results from models 1 and 2 are influenced by the high velocity components in the time-history signal. This is due to the fact that the VE properties used in the model are based only on one natural frequency of vibration. Therefore in the analyses using models 1 and 2, for a given velocity, the force will increase, because the damping coefficient used by the models will be larger than the actual damping value at that higher mode frequency. If the simpler models which use properties based on only one mode of vibration (models 1 and 2) are used to model the FCD for earthquakes, they will not correctly account for the higher modes and the maximum force will likely be larger than observed in tests.
CHAPTER 5: Numerical Modelling of Fork Configuration Dampers (FCDs)

Figure 5.21 Modelling FCD-B3: Landers 1992 Time-History of the 85-Storey Case Study

Figure 5.22 Modelling FCD-B3: Loma Prieta Time-History of the 85-Storey Case Study
5.4 FURTHER REFINEMENT OF MODEL 3 THROUGH ADDITIONAL CALIBRATION

Model 3 utilized parameters representing the damper and spring coefficients as described in Section 5.2.2. The parameters were determined using a nonlinear least squared regression analysis for a for a large number of temperatures and frequencies. Investigating the model 3 results, the maximum force is generally underestimated and the stiffness, damping and temperature-histories could be improved to match their time-history profiles.

The GMM4TC is very flexible and can be further calibrated to improve its accuracy. This could be done by including many more test data points for the regression analysis or by concentrating test data points near the expected loading case. Another method to calibrate the model could be to modify the parameters that contribute the most to the stiffness, damping and temperature response. To further calibrate the model, two of the terms in the modelling constants were refined. In particular the temperature correction factor, Eq. (5.39), was increased by a factor of two and $\beta_{0,ref}$ in Table 5-3, was adjusted to $\beta_{0,ref} = 0.263$. These two parameters were shown to best match the temperature, stiffness and damping response the best over a wide range of loading cases of interest.

Two examples of the effects of the further refinement of model 3 are shown in Figures 5.23 and 5.24. Figures b and c show that the maximum force is better predicted using the calibrated models compared to the uncalibrated response. Also the stiffness and damping coefficients of the calibrated model, shown in Figures d and e, are now closer to the experimental results over the full duration. Also, the temperature as shown in Figures f match the temperature measured with the ThermaCAM closer than the original as well.

An another example of improved FCD response to the Loma Prieta earthquake is shown in Figure 5.25. For this example the response is characterized by higher mode effects. Again the calibrated response is improved over the uncalibrated response.

Although the calibration attempts improved the modelling, there is further calibration that could be performed to attempt to match the stiffness and damping profiles more accurately. In particular in Figures e, the damping coefficient is slightly higher than the experiment for both models and the stiffness is less than the experiment for both models. A possible further calibration would be to reduce some of the terms that effect the damping $\beta_{0,ref}$ and $\psi_1$ through $\psi_4$ and reduce some of the terms that effect the stiffness $G_0$ through $G_4$. These attempts at calibration are meant to show that through altering the input modelling parameters or by adjusting the parameters used for the nonlinear least-squared regression analysis a more refined response can be achieved.
Figure 5.23 Calibrated Model 3: HSHC3 and a Frequency of 0.3 Hz at a Targeted Strain of 100% for 100 Cycles

Figure 5.24 Calibrated Model 3: USHC3 and a Frequency of 0.5 Hz at a Targeted Strain of 200% for 10 Cycles
5.5 MODELLING FCDS IN COMMERCIAL SOFTWARE

FCDs can be modelled in commercial software by using the equivalent VE method in shear with a frame element. The effective Kelvin-Voigt modelling properties are estimated using the techniques specified in Section 5.3.1. The FCD shear properties, $k_{FCD}$ and $c_{FCD}$, are used as the effective stiffness and damping properties at a rigid offset at the FCD midpoint in the local shear direction. All other directions in the FCD are made sufficiently rigid, to concentrate all deformation at the center of the clearspan. This modelling technique is valid for the wind response or to assess the global damping characteristics of the building. Also upper and lower bound properties could be used to effectively bound the design.

5.6 MODELLING CONCLUSION

Three models were developed and implemented in a FE program developed to model the VE material behaviour: i) the Kelvin-Voigt model (KVM), ii) the generalized Maxwell model (GMM4) and iii) the generalized Maxwell model with a temperature correction factor (GMM4TC). All models were compared to the harmonic VE material characterization tests. All compared well to the small scale test specimens.

The models were implemented in a FE program developed to model the FCD shear behaviour: i) the equivalent VE model (model 1) ii) the spring KVM model (model 2) and iii) the GMM4TC model (model 3). All models were compared to FCD harmonic characterization tests, wind time-histories and earthquake time-histories. The results have implications for two different loading conditions i) ambient vibrations or wind loading and ii) earthquake loading.
5.6.1 Model Implications for Practitioners

In this chapter, various models were proposed to model the VE material behaviour as well as the FCD behaviour for various loading scenarios. Table 5-4 provides a summary of all the models discussed in this chapter and their applicability for a given analysis case. The FCD models (models 1, 2 and 3) are the focus of this discussion, because the VE material models are built into these FCD models.

### Table 5-4 Comparison of Models

<table>
<thead>
<tr>
<th>Model Type</th>
<th>Variable Frequency</th>
<th>Temperature Dependent</th>
<th>Degrees of Freedom</th>
<th>Modelling in Commercial Software</th>
<th>Analysis Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>VE Material Models</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>KVM</td>
<td>No</td>
<td>No</td>
<td>1</td>
<td>Yes</td>
<td>Modal Damping, Wind, Seismic</td>
</tr>
<tr>
<td>GMM4</td>
<td>Yes</td>
<td>Yes</td>
<td>4</td>
<td>No</td>
<td>Yes, Yes, No</td>
</tr>
<tr>
<td>GMM4TC</td>
<td>Yes</td>
<td>Yes</td>
<td>4</td>
<td>No</td>
<td>Yes, Yes, Yes</td>
</tr>
<tr>
<td>FCD Models</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Model 1</td>
<td>No</td>
<td>No</td>
<td>1</td>
<td>Yes</td>
<td>Yes, Yes, No</td>
</tr>
<tr>
<td>Model 2</td>
<td>No</td>
<td>No</td>
<td>2</td>
<td>Yes</td>
<td>Yes, Yes, No</td>
</tr>
<tr>
<td>Model 3</td>
<td>Yes</td>
<td>Yes</td>
<td>5</td>
<td>No</td>
<td>Yes, Yes, Yes</td>
</tr>
</tbody>
</table>

- **Model 1** - The equivalent VE properties in shear are defined by the formulas given in Section 5.3.1, based on estimates of the assembly connection stiffness, the damping coefficient and the damping stiffness of the VE material. The equivalent damping and stiffness coefficients are developed at a single frequency, strain and temperature. As observed in the harmonic characterization and wind tests for samples FCD-A and FCD-B, the model is capable of predicting the harmonic behaviour well and the wind behaviour well, because these types of vibrations are characterized by response in predominantly one mode of vibration. Observing that the VEM heats up over time and applying conservative upper and lower design bounds for the temperature and strain levels, the model can be used for bounded wind analyses. Also, because it is a single degree of freedom model and can be modelled as a frame element in shear, it can be easily implemented within commercial software programs which can be used to obtain the added damping in a structure for a given mode of vibration.

For excitations that have multiple frequency components, model 1 is not effective. This was observed when comparing the modelling behaviour to the experimental behaviour for the earthquake time-histories. The force in the model was much higher than observed in the experiment, because the model is extremely sensitive to velocity fluctuations in the displacement signal from higher mode contributions.

It is recommended that model 1 be used for wind vibrations and the assessment of overall added damping provided by the FCDs and recommended for design.
ii) **Model 2** - Model 2 is a two degree of freedom model consisting of a spring (representing the FCD steel assembly shear stiffness) and Kelvin-Voigt model (representing the damping coefficient and the damping stiffness of the VE steel assembly) in series transformed to act in shear. Again the VE material properties can only be defined for a single frequency, strain and temperature. Similar to model 1, model 2 captured the harmonic characterization and wind test response well, because these types of vibrations are characterized by response in predominantly one mode of vibration. Like model 1, conservative design bounds are recommended for wind analyses and to assess the overall added damping.

Similar to model 1, model 2 is not able to capture multiple frequency responses well, because the model is again defined for only the single frequencies, strains and temperatures. This was observed in the earthquake time-histories, however model 2 is slightly less sensitive to the high velocity components than model 1.

For very large MCE type earthquakes, this model could be easily extended to model viscoelastic plastic behaviour by replacing the spring element with an elasto-plastic element, possibly within commercially available software, however the displacement signal would have to be mainly in a single mode of vibration.

ii) **Model 3** - Model 3 is a five degree of freedom model consisting of a spring in series with the generalized Maxwell model with the temperature correction factor (GMM4TC). This model is able to capture all types of load cases well, because its modelling parameters are determined based on a large range of frequencies and temperatures. Only the ambient temperature is required and the model computes the self-heating over the time-history duration. However, the parameters are based on a nonlinear regression analysis of the test data and may have biases towards certain frequencies or temperatures depending on the test data. With further calibration of the model parameters or selective test data, the model could be further refined. Also, the model tended to underestimate the force level slightly compared to the experimental data. Compared to models 1 and 2, model 3 was much less sensitive to high-velocity components in the response. Model 3 is a more robust model compared to models 1 and 2 because it is adequate for all loading time-histories. However it is more complicated and difficult to implement in current commercial software packages. The fractional derivative model could also be investigated and compared to the GMM4TC model to compare the self-heating, damping and stiffness response parameters.

It is recommended that model 3 be used for seismic analyses, because it can better capture the response over a large range of frequencies. If self-heating effects are considered important, it is the only model that is capable of capturing this aspect of the VE material response. Due to the complexity of the model it is unlikely that it will be made available for use in commercial software. To more accurately assess FCD forces it may be possible to apply displacement-histories obtained from commercial software and then apply the time-histories of FCD displacements that are computed and then applied to a single FCD
model where model 3 is implemented in order to get a more realistic assessment of the FCD forces.

Another technique that may serve as a valuable tool, would be to run a more detailed model to obtain an approximation of the self-heating effects, then, using the temperature data from the detailed model, a simpler model could be used to calculate the force-displacement hysteresis, such as model 1, where the spring and dashpot properties are estimated using the temperature data using the more detailed model.
CHAPTER 6: DESIGN OF HIGH-RISE BUILDINGS INCORPORATING FORK CONFIGURATION DAMPERS (FCDs)

6.1 DESIGNING HIGH-RISE BUILDINGS INCORPORATING FCDs

In this chapter, the previous chapters are built upon to develop an integrated design methodology for high-rise buildings equipped with FCDs. First a summary of the important FCD properties that are relevant to the different design loading cases are presented, followed by guidelines for wind design and seismic design of high-rise buildings with FCDs. Finally, a case study of an 85-storey residential condominium building located in downtown Toronto is used to highlight how the proposed design methodologies would apply to a real project.

6.2 WIND AND SEISMIC FCD DESIGN PHILOSOPHY

The FCD has two distinct response characteristics: i) viscoelastic response and ii) viscoelastic plastic response. A summary of design envelopes is presented in Figure 6.1. The viscoelastic FCD deformed shape is illustrated in Figure 6.1a. Within the viscoelastic region the built-up assembly responds linearly to the applied loads. The loading cases for which the FCD would respond in this range includes ambient vibrations, Service Limit State (SLS) wind storms up to a 10 year return period, Ultimate Limit States (ULS) wind storms up to a 50 year return period and Service Level Earthquakes (SLE) in seismic regions. This is necessary to ensure that the FCD responds linearly to relatively frequent design loads that will occur over the life of the building. In order to achieve this goal the maximum force, $F_{FCDf}$, can not exceed the factored 1 in 50 year ULS wind load. This can be expressed as a function of the FCD shear displacement using Eq. (3.6) and applying a safety factor of wind loads, 1.4, as:

$$F_{FCDf} = 1.4(k_{FCDuFCD}/\cos \theta)$$

(6.1)

Beyond this level, the viscoelastic FCD response will transition to viscoelastic plastic response with yielding of the connecting steel elements, as illustrated in Figure 6.1b. The design envelope for extreme and rare loading is illustrated in Figure 6.1c. For excursions into nonlinear regions it is important to have a ductile deformation mechanism. To ensure a ductile response, a capacity design is performed and a “fuse”
a) Viscoelastic Deformed Shape

b) Viscoelastic-Plastic Deformed Shape

c) FCD Design Hysteresis

Figure 6.1 FCD Design Summary

mechanism is selected. The “fuse” mechanism has two primary roles i) to prevent a brittle or non-ductile response, such as weld tearing or concrete crushing and ii) to protect the VE material from debonding or tearing. A desirable strength hierarchy is developed and brittle or limited ductility mechanisms are not permitted to occur as discussed in Section 4.3.1.1 and Section 4.3.2.1. The probable force of the FCD “fuse” mechanism \( F_{FCDpr} \), must be less than the factored resistance of any elements in series with the “fuse”, \( \phi R \). This can be expressed as:

\[
F_{FCDpr} < \phi R
\]  \hspace{1cm} (6.2)

Also, the VE material must not debond or tear prior to reaching the probable force of the FCD “fuse”, \( F_{FCDpr} \). To ensure this, the displacement capacity of the VE material, \( u_{VE\text{Max}} \), must exceed the displacement at the probable force, \( u_{VEpr} \). This can be expressed as (Kasai 2002):

\[
u_{VE\text{Max}} > u_{VEpr} = \frac{F_{FCDpr}}{k_{FCD}}
\]  \hspace{1cm} (6.3)
6.2.1 Proposed Wind Design Strategy with FCDs

Figure 6.2 provides a summary of the proposed wind design strategy for high-rise buildings incorporating FCDs. The full design strategy is described in detail in the following sections. Figure 6.2a is an illustration of the wind loading profile over the height of a rectangular building. The mean time-averaged wind, which increases up the height of the structure, strikes the face of the bluff body causing the structure to deflect statically as shown in Figure 6.2b. There is also a slow time-varying load in the along-wind direction, causing the building to deflect slowly. These loads do not cause resonant amplifications. The remainder of the building response in the along-wind direction is caused by wind turbulence, causing the structure to vibrate in one or more of its modes of vibration. This action induces inertial forces over the height of the structure. As the wind strikes the front of the bluff body, vortices are also shed off to the sides of the building, causing the structure to vibrate in one or more of its modes of vibration in the across-wind direction. Figure 6.2c illustrates the deformed shape at the maximum interstorey drift. The deformed shape of an elevation of coupled walls with FCDs and coupling beams is shown in Figure 6.2d. Shear deformation in the viscoelastic material is caused by the racking motion of the walls. The steel plates anchored in the left and right walls move relative to one another, as shown in Figure 6.2e. This action results in energy dissipation along the height of the structure, with damping present in all lateral modes of vibration. As discussed in Chapter 2, the addition of distributed damping decreases the dynamic wind-induced motion of the structure and reduces the large inertia loads up the height of the structure. Figure 6.2f displays the base moment time-history and the VE strain time-histories. The resulting VE material hystereses are shown as well and the hysteresis generated with the VE material Upper Bound (UB) and Lower Bound (LB) properties cycled to the design strains at the fundamental frequency in the along-wind direction. Figure 6.2g shows the same figures for the across-wind directions (base moment and strain-time history, VE material hysteresis and the hysteresis generated with the VE material UB and LB properties cycled to the design strains at the fundamental frequency).

There are two different design cases that are considered for a structure designed with FCDs: i) the global structural response and ii) the local FCD response.

i) Global Structural Response - When FCDs are introduced in RC coupled wall buildings replacing coupling lintel beams or outriggers, they alter the building structure stiffness and the building dynamic properties. The inclusion of the FCDs also affects the redistribution of stiffness between walls and beams, not only on the same storey, but also between adjacent stories. This can also cause redistribution of forces between adjacent lines of walls resisting loads in the same direction. The intended structural response is satisfactory if all design checks considered in Chapter 2 for high-rise design are satisfied. The SLS criteria is met if the 1/500 inter-storey drift levels and the 25 milli-g (for commercial buildings) and the 15 milli-g (for residential buildings) human perception criteria are met as defined by the NBC-2005. Sometimes
CHAPTER 6: Design and Analysis of High-Rise Buildings with Fork Configuration Dampers (FCDs)

FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

Figure 6.2 Proposed FCD Wind Design Summary

Design Cases
1. Drift Design
2. Adjacent Member Design

Figure 6.2 Proposed FCD Wind Design Summary

Design Cases
1. Wind Tunnel Properties
2. FCD Connection Design
torsional velocities are of concern, and a limit of $3 \text{milli-} \text{rad/s}$ for this response parameter have been set by the Council of Tall Buildings and Urban Habitat (Isyumov and Tschanz 2001). The drift limits are checked using FE models using the SLS effective static wind loads with appropriate cracked properties. The human perception criteria is checked by wind tunnel consultants with a High Frequency Base Balance (HFFB) wind tunnel study using dynamic properties obtained from the building designer. In addition, member strength checks are carried out for the forces induced by the ULS wind loading case. Of particular importance are the lintel beams in stories above or below the FCD locations due to the stiffness redistribution. Every member in the structure is modelled using the FE software with representative modelling properties and factored maximum member design forces are obtained for every member in the structure. All members are designed for strength using the Canadian Concrete Design Handbook CSA A23.3 (CSA 2004) or the Canadian Handbook of Steel Construction CAN/CSA S16-01 (CISC 2009).

ii) Local FCD Response - There are two possible failure modes for the FCDs, the first consisting of the tearing of the VE material and the second is the failure of the connecting steel members and anchorages. Maximum strain limits for the VE material for both the 1 in 10 year SLS (100% strain for across-wind and 125% strain for along-wind) and 1 in 50 year ULS (135% strain for across-wind and 170% strain for along-wind) are suggested for a desired stable performance. NSEC suggested a strain limit of 100% for the SLS across-wind load to ensure negligible VE material property degradation over the full wind load. This limit would preclude the VE ISD-111H material from suffering any deterioration under wind loads. The FCD VE material strains are checked using FE models with representative properties using SLS and ULS effective static wind loads. For each FCD, the connecting steel elements and the FCD-RC connection is capacity designed, ensuring a predictable failure mechanism with some redundancy if an extreme wind load were to occur. This is designed and checked using the CAN/CSA S16-01 (CISC 2009) and the CSA A23.3 (CSA 2004) by the capacity design methods described in the previous chapters. The full capacity design of FCD-A and FCD-B are presented in Appendix A. This design strategy ensures that the connections factored capacity in strength is larger than the factored force demand for all 1 in 50 year wind loads.

6.2.2 Considered Load Cases

During wind loading, buildings exhibit a three-dimensional response as defined in the previous chapters. The governing design cases are in the along-wind and across-wind directions, where the building and FCD have different response characteristics. In the along-wind, direction the building has a combination of a static and dynamic response, whereas in the across-wind direction, the building responds predominantly dynamically. Buildings also respond in torsion, which is mainly dynamic and therefore similar to the across-wind load case. The design loading is assumed to last for one hour at the peak loading levels.
i) **Along-wind Loading** - The SLS (1 in 10 year) and ULS (1 in 50 year) along-wind structure response is a combination of a time-averaged mean load, a slow varying background load not amplified by the dynamic behaviour of the structure and a dynamic inertia load that occurs when the structure vibrates in one or several of its natural modes of vibrations. As buildings increase in height they become much more dynamically sensitive and the inertia loads increase. Wind-tunnel consultants can provide an estimate of the contribution of dynamic inertia loads for a given structure (it can vary from 20% up to 80% of total inertia load, and this percentage increases with building height). The VE material stiffness, $G_E$, decreases with the frequency at which it is cycled, with a minimum value for static loads. The duration of the wind storm is assumed to last an hour at the peak load levels. Over the duration of the storm the VE material dissipates energy as heat, causing the VE material temperature to rise. The experiments presented in Chapter 4 showed that the increases in temperature where less than 5°C over an hour of sustained wind loading. The effect of added damping in the along-wind direction is less than in the across-wind direction.

ii) **Across-wind Loading** - The SLS (1 in 10 year) and ULS (1 in 50 year) across-wind structure response is completely dynamic. This consists of primarily the dynamic amplification of vibrations caused by vortices shed from the wind striking the front face of the building. These across-wind loads cause the structure to vibrate in one or more of its natural modes of vibration. The frequency content of the vortices shed off are often close to the fundamental frequency and can even cause the vortex shedding “lock-in” phenomenon. The inertia loads of very tall structures can sometimes govern the ULS strength design of the structural members. Over the duration of the storm the VE material dissipates energy as heat, increasing the VE internal temperature more than in the along-wind direction, because of the greater energy dissipation that takes place. The experiments showed increases in temperature of less than 7°C over an hour of across-wind loading, however this would be building and storm dependent. The effect of added damping in the across-wind direction are substantial, as discussed in previous chapters.

Table 6-1 provides a summary of the load cases described above.

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Design Case</th>
<th>Assumed Duration</th>
<th>Contribution to Total Strain as a Percentage</th>
<th>Temperature Rise (Building Dependent) (°C)</th>
<th>Targeted Design Maximum VE Strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Along-wind</td>
<td>SLS</td>
<td>1 hr</td>
<td>Building Dependent (20 to 80%) Building Dependent (20 to 80%)</td>
<td>Building Dependent 2 Building Dependent 4</td>
<td>125</td>
</tr>
<tr>
<td>Along-wind</td>
<td>ULS</td>
<td>1 hr</td>
<td>Building Dependent (20 to 80%) Building Dependent (20 to 80%)</td>
<td>Building Dependent 3 Building Dependent 5</td>
<td>175</td>
</tr>
<tr>
<td>Across-wind</td>
<td>SLS</td>
<td>1 hr</td>
<td>- 100</td>
<td>Building Dependent 3</td>
<td>100</td>
</tr>
<tr>
<td>Across-wind</td>
<td>ULS</td>
<td>1 hr</td>
<td>- 100</td>
<td>Building Dependent 6</td>
<td>135</td>
</tr>
</tbody>
</table>
6.2.3 Suggested FCD Design Properties

A limitation in the wind design of high-rise buildings as compared to earthquake design, is the reliance of structural engineers on the wind tunnel test results to determine the lateral design loads, load combinations and dynamic response prediction. This removes some of the knowledge of the critical dynamic response parameters from the structural designer. Also, the response of high-rise buildings is dynamic, however, linear elastic models with effective static loads obtained from the wind tunnel results are used for design. These effective static loads include large dynamic inertia loads. Unfortunately this leads the structural engineer to believe the load is similar to loads on low-rise structures, where there is little dynamic contribution. With the increase in use of Computational Fluid Dynamics, structural designers may be able to better understand the benefits or consequences of added structural damping on the wind response of high-rise buildings in the near future.

Due to the lack of understanding of wind, the reliance on wind tunnel results, it is very difficult to determine what the direction of attack is for any 3-D FE model that is used for the design of high-rise structures. Also the directionality of the wind governing the design may follow general trends, however determining the direction of the governing wind load is very difficult. The designer does not know if the load is caused by along-wind loading, by across-wind loading, loading caused by torsion or by loading at another angle of attack. The “effective storey-by-storey wind loads” and 24 load combinations obtained from the wind tunnel analysis is intended provide an effective wind load envelope covering the maximum possible force on every member in the FE model (BLWTI., 1999).

In the proposed design strategy it is recommended to use current linear elastic static FE modelling techniques and software as described in chapter 2, combined with assumptions of FCD properties. The design properties are based on relatively conservative assumptions in order to represent the worst possible design scenarios. Users of the FCD must estimate the steel assembly connection stiffness in shear and using the techniques described in the previous chapters to combine the VE Kelvin-Voigt model (KVM) and the steel assembly connection forming an equivalent VE model (model 1). This will result in a linearized effective Kelvin-Voigt model of the FCD. Alternatively, “best guess” property assumptions could be used to achieve the most likely response along with a factor of safety to account for uncertainties.

Because of the variability in the concrete material properties caused by cracking and of the VE material under different loading conditions, a bounded analysis is used. Note that although bounded analysis cases are used, the results from case studies show little difference between the upper bound and lower bound properties of the VE material for wind design, because the temperature rise is relatively small. Tables 6-2 and 6-3 list the bounded properties used for design. The four design cases considered are: i) wind tunnel studies, ii) drift design, iii) adjacent member design and iv) FCD connection design:
i) Wind Tunnel Studies - Dynamic building properties are required for the high frequency base balance test conducted by the wind tunnel consultants to determine the dynamic response of the building. The wind tunnel consultants typically ask for the frequencies and mode shapes of the first three primary modes of vibration and the corresponding critical damping ratios. For coupled wall structures sometimes these primary modes of vibration may be dynamically coupled (having the primary directional components in all three modes of vibration). The critical damping ratio due to the supplemental damping in the first three modes is obtained from the analytical results as an average of upper and lower bound properties at the structures frequencies of vibration. The lower bound VE material properties are assumed to occur at the end of a one hour wind storm, at a given frequency of vibration. It assumes that the structure is vibrating at the SLS design strain of $\gamma_{VE} = 100\%$ at an inherent temperature of $T_{VE} = 25^\circ C$. The upper bound VE material properties are assumed to occur at the beginning of a wind storm, when the material is “cold” at a lower bound estimate of the ambient room temperature of $T_{VE} = 20^\circ C$ for a given frequency of vibration. These modelling properties for the SLS across-wind load case are given in Table 6-2.

ii) FCD Connection Design Properties - Continuing with this conservative design approach, the largest linearized VE stiffness properties ($G_E$) at the structures natural frequency are used for the static FE model. This is assumed to occur at the beginning of a wind storm when the material is “cold” at $T_{VE} = 20^\circ C$. Another assumption will be that the response is 100% dynamic, as this will result in the worst case connection design force, at the design strain of $\gamma_{VE} = 135\%$. This should result in an FCD element with a relatively large stiffness, which using the equivalent FE modelling technique described in Chapter 3 should result in the largest possible connection force as evaluated by all 24 load combinations. This load is assumed to act when the structure is at the maximum displacement. Therefore to account for the chance that this could be caused by an across-wind 100% dynamic vibration, the maximum load is divided by the phase lag, $\cos \theta_{FCD}$, as described for by Eq. (3.11). Also, the ULS wind safety factor of 1.4 is used to ensure the connection remains linear for all wind loads. The ULS 1 in 50 year factored connection design force is:

$$F_{FCDf} = 1.4(k_{FCD}u_{FCDMax}/\cos \theta_{FCD})$$  \hspace{1cm} (6.4)

These modelling properties for the upper bound ULS across-wind properties and tabulated in Table 6-2.

iii) Drift Design - The worst case scenario for the drift response would be at the end of SLS storm in the along-wind direction when the VE materials at their maximum temperature. These are the lower bound VE material properties. The wind tunnel consultant must give an estimate for what percentage of the maximum response is dynamic, ($\%$dynamic), and of what percentage of the response is static, ($\%$static). The static stiffness, $G_S$, is defined at the end of the actual SLS along-wind tests. VE material properties at $T_{VE} = 25^\circ C$ at the fundamental frequency, at a strain of ($\%$dynamic/100%)125% are used. After both the static FCD stiffness, $k_{FCD_S}$, and dynamic FCD stiffness, $k_{FCD}$, are calculated, the effective FCD
stiffness, $k_{\text{FCD}_{\text{eff}}}$, used in the static FE analysis is calculated as:

$$k_{\text{FCD}_{\text{eff}}} = (%\text{dynamic}/100\%)k_{\text{FCD}} + (%\text{static}/100\%)k_{\text{FCD}_{\text{S}}} \quad (6.5)$$

These linearized modelling properties are the lower bound SLS across-wind properties tabulated in Table 6-3.

iv) Adjacent Member Strength Design - The adjacent member strength design worst case scenario would be at the end of the ULS wind storm in the along-wind direction. This is of particular importance to the beam members directly above or below the FCD locations. Again an assumption defining the percentage of the response that is dynamic, (%dynamic), and the percentage of the response that is static, (%static) must be made. The static stiffness, $G_S$, is defined at the end of the actual ULS along-wind tests. VE material properties at $T_{VE} = 30^\circ C$ at the fundamental frequency, at a strain of (%dynamic/100%)170%. After both the static FCD stiffness, $k_{\text{FCD}_{S}}$, and dynamic FCD stiffness, $k_{\text{FCD}}$, are calculated, the effective FCD stiffness, $k_{\text{FCD}_{\text{eff}}}$, to be used in the static FE analysis is calculated as:

$$k_{\text{FCD}_{\text{eff}}} = (%\text{dynamic}/100\%)k_{\text{FCD}} + (%\text{static}/100\%)k_{\text{FCD}_{\text{S}}} \quad (6.6)$$

These equivalent FCD modelling properties are the lower bound ULS across-wind properties that are tabulated in Table 6-3.

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Design Bound</th>
<th>Strain</th>
<th>VE Temperature</th>
<th>$\gamma_{\text{Max}}$(%)</th>
<th>$T_{VE}(^\circ C)$</th>
<th>$f(Hz)$</th>
<th>$G_E(MPa)$</th>
<th>$\eta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Across-wind SLS</td>
<td>Upper Bound</td>
<td>±100</td>
<td>20</td>
<td>0.123 0.191 0.327</td>
<td>0.82 0.96 1.12</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Lower Bound</td>
<td>±100</td>
<td>25</td>
<td>0.101 0.150 0.250</td>
<td>0.72 0.86 1.02</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Across-wind ULS</td>
<td>Upper Bound</td>
<td>±135</td>
<td>20</td>
<td>0.120 0.188 0.323</td>
<td>0.82 0.94 1.08</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Lower Bound</td>
<td>±135</td>
<td>30</td>
<td>0.075 0.108 0.167</td>
<td>0.62 0.77 0.91</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Design Bound</th>
<th>Strain</th>
<th>Temperature</th>
<th>$\gamma_S \pm \gamma_{\text{Max}}$(%)</th>
<th>$T(^\circ C)$</th>
<th>$G_E(MPa)$</th>
<th>$G_T(MPa)$</th>
<th>$\eta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Along-wind SLS</td>
<td>Upper Bound</td>
<td>75±50</td>
<td>20</td>
<td>0.023 0.194 0.327</td>
<td>0.81 0.98 1.17</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Lower Bound</td>
<td>75±50</td>
<td>25</td>
<td>0.018 0.156 0.254</td>
<td>0.705 0.865 1.05</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Along-wind ULS</td>
<td>Upper Bound</td>
<td>100±70</td>
<td>20</td>
<td>0.020 0.193 0.327</td>
<td>0.82 0.97 1.15</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Lower Bound</td>
<td>100±70</td>
<td>30</td>
<td>0.016 0.113 0.178</td>
<td>0.60 0.75 0.92</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
6.2.4 Wind Design Procedure with FCDs

The suggested wind design procedure for high-rise buildings with FCDs is summarized in the flowchart shown in Figure 6.3. The steps for the high-rise design with FCDs are outlined below:

**STEP 1)** The structural layout including the FCDs is established with the architect and structural engineer.

**STEP 2)** A preliminary lateral load resisting system with the FCDs is designed with the structural engineer. The structural engineer designs the concrete walls and coupling beams (wall widths and lengths, coupling beam depths, reinforced concrete strength and preliminary reinforcing profile up the height of the building). The targeted added damping is established and a redesign is carried out by determining the optimum number of FCDs and their locations based on the initial building property estimates.

**STEP 3)** SLS and ULS 3-D FE models are created of the building lateral load resisting system including FCDs modelled using properties from the equivalent VE model (model 1) corresponding to each loading case.

**STEP 4)** The drifts from the design strain of FCD material are checked under the SLS wind loads.

**STEP 5)** Based on the ULS factored wind loads, the member ultimate strength limits are checked. The strength limits of the FCD connection and FCD strain limits are checked.

**STEP 6)** A wind tunnel study is conducted using the average SLS VE material properties. The acceleration and torsional velocity response is obtained from the wind tunnel study. Also, new loads and load combinations are obtained from the wind tunnel study.

**STEP 7)** Steps 3 to 5 are conducted again with new wind tunnel loads.

If any of the design requirements are not met a new iteration is carried out by altering the number, location and properties of the FCDs or the lateral load resisting system.
CHAPTER 6: Design and Analysis of High-Rise Buildings with Fork Configuration Dampers (FCDs)

Figure 6.3 Proposed Wind Design Procedure of RC Coupled Wall High-Rise Buildings with FCDs
6.2.4.1 Extreme Wind Loads

Performance-based wind engineering has recently been looked at for special structures subject to unpredictable loading conditions (Bracci 2008). As large design based wind storms become more frequent and unpredictable due to climate change (Steenbergen et al. 2009, Kareem 1988), structures require redundant robust lateral load resisting systems to safely resist such loads. Also, the upstream terrain can change significantly over the service life of the building. Compared to vibration absorbers, added distributed viscous damping has a much better performance over all ranges of loadings since the distributed viscous damping will always be available even if loading conditions or structural properties change. TMDs are usually optimized at the design stage to reduce the dynamic response due to the 1 in 10 year wind loads. As such, when the building is subject to other larger loads and more rare situations, the vibration absorber may not be useful, because of improper tuning of the damper itself.

6.3 PROPOSED SEISMIC DESIGN STRATEGY WITH FCDS

6.3.1 Proposed Seismic Design Strategy with FCDs

Figure 6.4 provides a summary of the proposed design strategy of high-rise buildings with FCDs for seismic loading. The seismic design of high-rise buildings with FCDs is based on the well known principles of capacity design for the global building response and also for the local FCD response. In order to maintain these requirements a “fuse” mechanism needs to be introduced into the FCD for two important reasons: i) to ensure the force introduced into the walls by the VE material does not exceed design limits and potentially introduce unexpected and unpredictable failure mechanisms and ii) to ensure the VE material does not fail. The “fuse” mechanism designed for FCD-B and tested in full-scale consisted of two Reduced Beam Sections (RBS) in the built-up assemblies connecting the FCDs into the concrete walls. However alternative reliable and predictable ductile mechanisms such as shear yielding of the connecting elements, anchorage yielding or friction slip in the connections could also be used.

Performance is judged from the expected response of the structure to two levels of earthquakes that are prescribed by the Performance Based Seismic Design Guidelines (SEAONC 2007, CBTUH 2008, LATBSDC 2008 and PEER 2009) as was described in Chapter 2. These two design critical excitation levels, are i) the Service Level Earthquake (SLE) and ii) the Maximum Credible Earthquake (MCE) are described below:

i) Service Level Earthquake (SLE) -  When subjected to frequent earthquakes (roughly 50 year return periods) high-rise buildings equipped with FCDs are expected to perform better than the same structure with RC coupling beams with diagonal reinforcing patterns, conventional reinforcing patterns or structural steel coupling beams. This is due to the distributed damping that is added to all of the lateral modes of vibration and the subsequent reduction in to inertia loads. The intended range of FCD response under
CHAPTER 6: Design and Analysis of High-Rise Buildings with Fork Configuration Dampers (FCDs)

Fork Configuration Dampers (FCDs) for Enhanced Dynamic Performance of High-Rise Buildings

Figure 6.4 Proposed FCD Seismic Design Summary

- a) Earthquake Loading
- b) Deformed Shape at Max Interstorey Drift
- c) Elevation View Deformed Shape for Large Earthquakes
- d) FCD Deformed Shape for Large Earthquakes
- e) Viscoelastic FCD Behaviour
- f) Viscoelastic-Plastic FCD Behaviour
- g) Modelling Techniques (Linear Elastic “Fuse”)
- h) Modelling Techniques (Elasto-Plastic “Fuse”)

Energy Dissipation:
1. VE Material
2. FCD “Fuse” Activation
3. Coupling Beam Yielding
4. Base Hinge Yielding

Fork Configuration Dampers (FCDs) for Enhanced Dynamic Performance of High-Rise Buildings
these frequent earthquakes is shown in Figure 6.4e. The FCDs “fuse” connection is designed and expected to remain within the linear elastic range with a factor of safety for these more frequent earthquakes. The shear behaviour of the FCD during this level of excitation is the same as the expected response to wind loads, and is effectively a viscoelastic response. This can be modelled in FE software as a Kelvin-Voigt model in shear, used to represent the VE material behaviour, in series with a spring model also in shear, representing the linear elastic behaviour of the steel elements as illustrated in Figure 6.4g. It is expected that the only substantial energy dissipation is from the VE material undergoing shear deformations in this loading case.

ii) Maximum Credible Earthquake (MCE) - Subject to extremely rare earthquake (about a 1 in 2500 year return period) high-rise buildings with FCDs are expected to perform significantly better than the same structure with conventional coupling methods (RC coupling beams with diagonal reinforcing patterns, conventional reinforcing patterns or structural steel coupling beams). When large deformations are introduced to the FCD, the force limiting “fuse” provides a limit to the shear forces, effectively limiting the FCD forces transferred through the connection to the walls and the VE material shear strains. This response is termed a “viscoelastic-plastic” characterized by a “cats-eye” hysteretic response in shear as shown in Figure 6.4f. This response can be modelled using an elasto-plastic element in series with the Kelvin-Voigt model for the VE material and is termed the “viscoelastic-plastic model”.

At the base of the structure the walls are expected to reach their flexural capacity and form flexural hinges at the base. Coupling beams (if any) along the same line of perforations with the FCDs would be expected to yield in shear or flexure. The design of these adjacent elements would be the same as required of a traditional coupled wall structure as described in Chapter 2.

The shear deformation at the yield force for a traditionally or diagonally reinforced coupling beam is expected to be smaller than the yield deformation of the FCD steel elements. Also, since the FCDs add distributed damping, the inertia loads are expected to be reduced. For these reasons the FCDs would be expected to reduce the damage compared to a traditionally designed coupled wall building.

With proper design of the “fuse” elements the total shear deformation capacity of the FCD assembly is larger compared to the deformation capacity of conventional beams. This was achieved in test specimen FCD-B using the reduced beam section detail in the connections. Other “fuse” connections such as a shear yielding link, anchorage yielding, bending or friction connections are also possible. For best performance the “fuse” should be very stiff to engage the VE material effectively and have a flat yield plateau with a stable and predictable cyclic behaviour.

Another possible connection could be a replaceable connection detail as described in Chapter 3, where traditional “fuse” elements are used with a bolted or post-tensioned connection. After a large
earthquake the “fuse” element is expected to be activated and if significant damage was observed, the anchorage connection is released and a new FCD is installed. This would represent a significant decrease in rehabilitation time of the building after an earthquake, by eliminating the difficult repairs to coupling beams that would be required as observed in the high-rise reconnaissance work after the Conception Earthquake in Chile (LATBSDC 2010).

Additional benefits could be observed at the beginning of the project as well. For example constructing the walls separately with special anchorage points would provide for easy installation of the FCDs up the height of the walls after the walls have been cast. Also, this could eliminate differential wall settlement forces.

6.3.2 Modelling FCDs for Earthquake Analyses

In current FE analysis software the ability to model using a spring generalized Maxwell model with temperature corrections is not feasible. However software packages are moving towards including damping elements that can model frequency dependent properties. To model nonlinear behaviour in the FCD, the spring is now modelled as an elasto-plastic element, shown in Figure 6.4h. The viscoelastic-plastic modelling technique was described briefly in Section 3.4.4 If the elasto-plastic element is not expected to yield, the behaviour can be modelled as the “equivalent viscoelastic model” with frequency dependent VE properties as shown in Figure 6.4g. A nonlinear time-history analysis with the frequency dependent properties of the VE material, will capture the full elasto-plastic response for the large earthquakes, while still capturing the equivalent viscoelastic response under small amplitude vibrations.

The strategy for modelling the VE material properties for earthquakes is different than for the wind vibrations, because of the shorter duration of a typical earthquake and the frequency content of the vibrations. Fan (1998) suggests to use an average or ambient temperature for linearized modelling properties for the given mode of vibration. A recommended value of $T_{FE} = 23^\circ C$ is used for the linearized material properties. Since there are many modes that can be active in an earthquake, it would be beneficial if commercial software had the ability to offer frequency dependent properties to account for the VE material being excited at different speeds, however currently there are none available. If there is no ability to capture the frequency dependency of the VE material, then the fundamental mode of vibration linearized properties are recommended, since that is the predominant response. The consequence of this is that the damping coefficients at lower frequencies are larger compared to the damping coefficients at higher frequencies and therefore the viscous response component will be unrealistically amplified. This modelling approach is described in Section 5.3.4.3.2. Table 6-4 displays properties for different strain amplitudes and for a range of frequencies.
There are three levels of earthquake ground motions that are considered as defined in ASCE 7-05 (ASCE 2005) and the high-rise building Performance Based Seismic Design Guidelines (SEAONC 2007, CBTUH 2008, LATBSDC 2008 and PEER 2009). The intended response characteristics for the i) Service Level Earthquake, ii) Design Level Earthquake and iii) Maximum Credible Earthquake are described below:

### i) Service Level Earthquake
- The FCD “fuse” remains linear and the majority of the energy dissipation is caused by VE material shear deformations. The targeted maximum strain is $\gamma_{\text{VE,max}} = 150\%$. A temperature rise of less than $2^\circ C$ is expected and the expected duration of sustained loading is 1 minute. There is no expected yielding of any members, however, this needs to be checked. The added damping should help reduce the building motion and damage to nonstructural elements as well. If the inclusion of FCDs causes stiffness discontinuities it is possible that minor cracking could occur in the concrete coupling beams directly above or below the FCD locations.

### ii) Design Level Earthquake
- For the DLE earthquake the FCD “fuse” could remain in the linear range or could activate resulting in a viscoelastic-plastic response. A capacity design procedure is used to ensure that the VE material does not fail and that large force amplifications do not occur in the walls. If the fuse is designed properly, the strain limit of $\gamma_{\text{VE,max}} = 400\%$ should not be exceeded. The expected temperature rise is of less than $4^\circ C$ and the expected duration of the sustained loading is 1 minute and 30 seconds. There could also be yielding of coupling beams and wall hinging at the base, however it is intended to be minimal. The distributed viscous damping would help to delay or reduce damage in the structural members and nonstructural elements by controlling the maximum deflections. If there are stiffness discontinuities caused by the inclusion of the FCDs there may be more damage at locations directly above or below the FCDs. If this is the case the stiffness will be redistributed as a results of the overloading of these elements.

### iii) Maximum Credible Earthquake
- For the MCE earthquakes the FCD “fuse” would activate in the nonlinear range. If the fuse is designed properly, the strain limit of $\gamma_{\text{VE,max}} = 400\%$ should not be exceeded. There is an expected temperature rise of less than $5^\circ C$ and the expected duration of sustained loading is 2 minutes. There would also be yielding of coupling beams and wall hinging at the base. The distributed damping would reduce damage in the structural members and the

<table>
<thead>
<tr>
<th>Design Strain</th>
<th>VE Temperature</th>
<th>$f(\text{Hz})$</th>
<th>$G_E(\text{MPa})$</th>
<th>$\eta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_{\text{VE,max}}$ (%)</td>
<td>$T_{\text{VE}}(^\circ \text{C})$</td>
<td>0.1</td>
<td>0.3</td>
<td>1</td>
</tr>
<tr>
<td>±50</td>
<td>23</td>
<td>0.113</td>
<td>0.170</td>
<td>0.282</td>
</tr>
<tr>
<td>±100</td>
<td>23</td>
<td>0.110</td>
<td>0.166</td>
<td>0.281</td>
</tr>
<tr>
<td>±200</td>
<td>23</td>
<td>0.102</td>
<td>0.158</td>
<td>0.271</td>
</tr>
<tr>
<td>±400</td>
<td>23</td>
<td>0.087</td>
<td>0.142</td>
<td>0.251</td>
</tr>
</tbody>
</table>
nonstructural elements by controlling the maximum deformation. If there are stiffness discontinuities caused by the inclusion of the FCDs there may be more severe damage in locations directly adjacent to the FCDs, but the loads would redistributed to all other structural members.

A summary of the expected response characteristics of the buildings with FCDs subject to earthquakes is given in Table 6-5.

**TABLE 6-5 Description of Load Cases**

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Design Case</th>
<th>Duration</th>
<th>Contribution to Total Strain as a Percentage</th>
<th>Temperature Rise (°C)</th>
<th>Targeted Maximum VE Strain (%)</th>
<th>“Fuse” Activation Analysis Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earthquake</td>
<td>SLE</td>
<td>60s</td>
<td>-</td>
<td>100</td>
<td>2</td>
<td>150</td>
</tr>
<tr>
<td>Earthquake</td>
<td>DBE</td>
<td>90s</td>
<td>-</td>
<td>100</td>
<td>4</td>
<td>400</td>
</tr>
<tr>
<td>Earthquake</td>
<td>MCE</td>
<td>180s</td>
<td>-</td>
<td>100</td>
<td>5</td>
<td>400</td>
</tr>
</tbody>
</table>

The ultimate FCD shear properties of FCD-A and FCD-B are summarized in Table 6-6 based on the full-scale test results. The connection strengths could be used to model the elasto-plastic behaviour in a viscoelastic-plastic model if the same FCD assembly and connection is used. These design properties are illustrated graphically in Figure 6.5.

**TABLE 6-6 Design Shear Connection Parameters**

<table>
<thead>
<tr>
<th>Factored Yield Strength</th>
<th>Nominal Yield Strength</th>
<th>Ultimate Strength</th>
<th>Maximum VE Material Displacement</th>
<th>Total Available Assembly Displacement</th>
<th>Total Available FCD Displacement</th>
<th>Total Available FCD Chord Displacement</th>
<th>Total Available FCD Chord Rotation</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_y (kN)$</td>
<td>$F_y (kN)$</td>
<td>$F_u (kN)$</td>
<td>$u_{VE} (mm)$</td>
<td>$u_A (mm)$</td>
<td>$u_{FCD} (mm)$</td>
<td>$\theta_{FCD} (rads)$</td>
<td></td>
</tr>
<tr>
<td>FCD-A</td>
<td>607</td>
<td>675</td>
<td>1366</td>
<td>20</td>
<td>22</td>
<td>44</td>
<td>0.028</td>
</tr>
<tr>
<td>FCD-B</td>
<td>600</td>
<td>667</td>
<td>1331</td>
<td>20</td>
<td>100</td>
<td>120</td>
<td>0.057</td>
</tr>
</tbody>
</table>
Figure 6.5 Proposed FCD Design Hysteresis
6.3.3 Seismic Design Procedure with FCDs

The suggested seismic design flowchart for buildings incorporating FCDs is shown in Figure 6.6 and is outlined below:

**STEP 1)** The functional layout is established with FCDs and capacity design approach is established with the architect, owner and structural engineer.

**STEP 2)** A preliminary lateral load resisting system with FCDs is established by the structural engineer. The structural engineer designs the concrete walls and coupling beams (wall widths and lengths, coupling beam depths, reinforced concrete strength and preliminary reinforcing profile up the height of the building). The targeted added damping for wind is established and a FCD is designed based on FCD locations and initial building property estimates.

**STEP 3)** Using commercial software the SLE and MCE 3-D FEM lateral analysis models including FCDs are created. The finalized FCD modelling parameters are established optimizing the FCD locations using techniques that were presented in Chapter 5.

**STEP 4)** SLE Assessment: The drift and acceleration limits of the building and design strains for the FCD material are checked. Also, it is verified that all members including the FCD connections remain essentially elastic.

**STEP 5)** DLE Assessment: The deformation capacity and strength degradation is checked for walls, coupling beams and FCDs, with a goal of immediate occupancy. The strength limits of all other members are checked. The VE material is checked to ensure it remains within design limits.

**STEP 6)** MCE Assessment: The deformation capacity and strength degradation is checked for walls, coupling beams and FCDs. The strength limits of all other members are checked. The non-structural elements are also checked to ensure they can handle the nonlinear deformations. The VE material is checked to ensure it remains within design limits.

**STEP 7)** The design is reviewed by the peer review panel.

If any of the design requirements are not met, a new FCD profile and lateral load resisting system must be considered and the process re-iterated.
FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

Figure 6.6 Proposed Seismic Design Procedure of RC Coupled Wall High-Rise Buildings with FCDs
6.4 85-STOREY CASE STUDY STRUCTURE

This sample structure was originally designed as a 75-storey residential condominium development in downtown Toronto by structural engineering firm Halcrow Yolles (HY). The development consists of 6 basements beneath a 5-storey retail podium followed by 70-storeys of residential condominiums. There are three primary tower floor plates with setbacks over the height of the building. The lateral system consists of a reinforced concrete coupled wall system with a mega-core at the elevator and at the staircase service core. There are 6 primary wing walls extending from the mega-core to the perimeter of the floor plates. The concrete strength in the mega-core is $f'_c = 80\text{MPa}$ and the wall thicknesses are $t_w = 1, 200\text{mm}$ at the base of the structure, decreasing to $f'_c = 40\text{MPa}$ and $t_w = 400\text{mm}$ at the top of the structure. The mega-core is continuous through the height of the structure. The residential tower sits on a post-tensioned 2.5$m$ thick transfer slab at the fifth floor, which transfers the gravity load from the columns and wing walls from the condominium to additional vertical members at levels below. This minimizes the number of vertical elements in the retail podium levels.

Wind tunnel studies on the tower were conducted by a wind tunnel consultant and the lateral load resisting system design was approved for the design height. The peak torsional velocities at the 1 in 10 year wind return periods were predicted to be right at the limits provided by the Council on Tall Buildings and Urban Habitat (CTBUH, Isyumov and Tshanz 2001). The developer became interested in the option of increasing the number of residential units at the top of the structure. The developer had sold a number of the units with the current lateral load resisting system and therefore the lateral system could not be altered to increase the height of the structure. The developer requested a re-design of the building to a new height of 85-storeys with a vibration absorber with no changes in the lateral load resisting system. The increased sellable space of the ten additional storeys, without a change to the wall thicknesses or the structural layout, represented significant additional sales revenue for the developer.

The building was redesigned with minimal changes in the structural configuration by the structural engineers assuming a vibration absorber was located at the top storey of the structure. It was assumed that a Tuned Mass Damper (TMD) would add 1% damping in each of the first three modes of vibration (X-sway, Y-sway and Torsion). The assumed inherent damping was estimated as 2% in the first three modes of vibration by the designers. The three floor plates of the structure for this alternate design are shown in Figure 6.7 and the concrete properties over the height of the building is also given in Table 6-7.
The structural consultants and vibration absorber consultants worked separately on their respective designs. The structure mode shapes not including the vibration absorber are shown in Figure 6.8 and in Table 6-8. The structural engineers made the structure as stiff as possible and the vibration absorber design was finalized with two-TMDs located at a radial distance from the core. One TMD was designed to add damping in the first X-sway mode of vibration, and the other to the first Y-sway mode of vibration. Acting together they were designed to add damping in the torsional mode of vibration. The TMD design was tuned and optimized for the 1 in 50 year ULS wind loads under the ULS stiffness conditions that were provided by the structural consultant.
The wind loads were determined from a high frequency force balance (HFFB) wind tunnel study conducted on the 85-storey structure as shown in Figure 6.9. The wind tunnel consultants provided some preliminary guidelines to assess the benefits of added damping on this particular structure. These guidelines were modified using techniques explained in Section 2.4 and the effects of the building inherent damping on the SLS dynamic response and total base moments as obtained by the information provided by the wind tunnel study shown in Figures 6.10 and 6.11, respectively. The circles on the graph represent the structural response levels without the TMD. The triangles represent the structural response levels with the TMD.
blue coloured triangles and circles represent the structural response levels of the building with an inherent damping of 1%, which is more representative of the in-situ measurements that have recently been measured on buildings with similar construction and height. The green coloured triangles and circles represent the building with the 2% inherent damping value that was assumed in the design. As can be seen in the figures, the acceleration levels of the structure would be just above the design limits if the lower damping values were used.

Figure 6.9 Wind Tunnel Study

Figure 6.10 SLS 1 in 10 year Dynamic Response as a Function of Damping
6.4.1 FCD Solution

Working closely with the structural consultant, a separate study was conducted to design the structure using FCD-As in place of reinforced concrete coupling beams. The design properties that were used were obtained from the test data and the design properties defined in Chapter 4 and the equivalent VE modelling technique that was described in Chapter 5. Two solutions, Option 1 and Option 2 are described below:

**Option 1:** Option 1 consists of the minimum number of FCD-As that were needed to achieve a minimum added damping of 1% in all modes of vibration. This 1% of added damping in all modes of vibration is the same performance level as the two-TMDs solution that was described above. The FCD was designed within the strict geometric tolerances set by the architect and had to fit within the current coupling beams. This affected the number and layer area of VE material that could be used and therefore the VE material stiffness. The depth of the connecting I-sections was also restricted, thus affecting the stiffness, \( k_d = 280kN/mm \), of the built-up steel assembly connection. For this reason the FCD stiffness is lower than what it may have been without these geometric constraints. As such, the concrete coupling beams above and below were replaced with structural steel W-sections, which reduce the stiffness of these adjacent members and increases the VE material deformations. The schematic of the two-FCD-A configuration is shown in Figure 6.12. There are a total of two-FCD-As at 122 locations (96 locations in the X-direction L18-L46 and 26 locations in the Y-direction L22-L34). The layout is shown in Figure 6.13. The concrete beams above and below the FCD locations are replaced with two class 1 structural steel W-sections in parallel cast directly into the concrete walls. The W-sections are anchored as proposed by Harries et al. (1993). The stiffness properties of the W-sections are tapered up the height of the building, as shown in Figures 6.13b and 6.13c.
Option 2: Option 2 is an alternative to Option 1 if the strict architectural requirements were slightly relaxed. This would represent a solution for other high-rise building markets with slightly larger floor-to-ceiling clearspans. The Option 2 design has one larger damper as opposed to two smaller dampers. There are 40 layers of 500 mm wide by 500 mm long by 6.5 mm thick viscoelastic material (VEM 111H) with a deeper assembly connection anchorage. The FCD used in Option 2 is labelled FCD-A2. This damper is significantly stiffer, both because of the increased size and number of VE layers, but also because of the increased stiffness of the built-up anchorage assembly which will be significantly taller. The assembly stiffness assumed in the design was $k_d = 2800 kN/mm$. This allowed for a more even stiffness distribution between the FCDs and concrete elements. This also reduces the stiffness irregularities at the FCD and RC coupling beam transitions. For these reasons, Option 2 does not require any change to the RC coupling beam designs in the structure. There are a total of 104 FCD-A2s in the same coupling beam locations in plan view as Option 1 (72 locations in the X-direction L17-L42 and 32 locations in the Y-direction L21-L36). The layout is shown in Figure 6.14. The FCDs are used in parallel with the current concrete beam layouts as shown in Figures 6.14b and 6.14c.
6.4.2 Evaluation of Added Damping

There are three methods that can be used to assess the global added damping provided by the addition of FCDs to a high-rise structure, which are described in the following. Using ETABS, a linear damping frame element is assigned the equivalent VE stiffness and damping properties in shear. The properties are determined using the equivalent VE model (model 1) and formulas from Section 5.3.1. The axial and bending properties are made sufficiently rigid to ensure the deformations are concentrated in the shear direction. Within the software, the damping matrix, $[C]$, is formed with the damping coefficients provided by all the FCDs elements, $c_{FCD}$. If the $[C]$ matrix can be decoupled using the mode shapes, the modal damping coefficient, $C_i$, in the $ith$ mode shape, $\{\phi_i\}$, can be obtained:
The damping ratio, $\xi_i$, in the $i^{th}$ mode of vibration is calculated as:

$$\xi_i = \frac{C_i}{2\omega_i M_i}$$

where $M_i$ is the modal mass and $\omega_i$ is the circular frequency in the $i^{th}$ mode of vibration.

Additionally, the damping can be obtained using a free vibration analysis with the logarithmic decrement technique. The logarithmic decrement in the $i^{th}$ mode of vibration, $\delta_i$, is defined as the natural logarithm of the ratio of any two successive peak amplitudes, $(u_n)_i$ and $(u_{n+1})_i$, in free vibration (Chopra 2003 and Clough and Penzien 2003):

$$\delta_i = \ln \left( \frac{(u_n)_i}{(u_{n+1})_i} \right) = \frac{2\pi \xi_i}{\sqrt{1 - \xi_i^2}}$$

In order to avoid picking up higher mode effects in the free vibration analysis, diaphragm storey displacements at about 2/3 up the height of the building are used for the two successive peak amplitudes. When the level of damping is low the critical damping ratio in the $i^{th}$ mode of vibration is approximately:

$$\xi_i = \frac{\delta_i}{2\pi}$$

To improve the accuracy, more cycles, $p$, are used, to calculate the logarithmic decrement:

$$\delta_i = \frac{1}{p} \ln \left( \frac{(u_n)_i}{(u_{n+p})_i} \right)$$

These two techniques are easily implemented, but are not useful for determining the most effective FCD placement is. As described in Section 3.4.5, an equivalent viscous damping technique can be used which allows the designer to identify where the optimal FCD locations through calculation of the total energy dissipated by $N$ FCDs for the $i^{th}$ mode:

$$E_{VEDi} = \sum_{j=1}^{N} c_{FCD(f)} \omega_j u_{FCD0ij}^2$$

The terms in Eq. (6.12) which contribute to the energy dissipation in the $i^{th}$ mode of vibration are the damping coefficient, $c_{FCD(f)}$, the number of dampers, $N$, and the maximum damper shear displacement, $u_{FCD0i}$. The equivalent viscous damping can be expressed as:

$$\xi_{eqi} = \frac{1}{4\pi} \frac{E_{VEDi}}{E_{Si}}$$
where $E_{Si}$ is the strain energy at the maximum displacement of the $i^{th}$ mode of vibration and is expressed as (Chang et al. 1991 and Constantinou and Symans 1992):

$$E_{Si} = \frac{1}{2} \phi_i^T [K] \phi_i$$  \hspace{1cm} (6.14)

and $\phi_i$ is the $i^{th}$ mode of vibration and $[K]$ is the stiffness matrix.

The damping ratio was computed using the modal damping equations, Eqs. (6.7) and (6.8) in ETABS. Additional checks were carried out by conducting free vibrations at the modes of interest using the logarithmic decrement technique using Eqs. (6.10) and (6.11). Some of the free vibration results will be shown in Section 6.4.2.1.3.

### 6.4.2.1 SLS Design

#### 6.4.2.1.1 Human Perception Criteria

The SLS modal properties were determined using a 3-D FE model with effective stiffness assumptions for the concrete elements. The average modal properties using the upper and lower bound SLS FE models were provided to the wind tunnel laboratory to determine the wind loads, accelerations and torsional velocities. The properties of the damper are determined for the first three modes of vibration and summarized in Tables 6-9 and 6-10, for 2-FCD-A and 1-FCD-A2 respectively. The methodology to determine the properties was described in Section 6.2.3.

### TABLE 6-9  Two-FCD-A: SLS Across-wind Dynamic Properties for First Three Modes of Vibration

<table>
<thead>
<tr>
<th>Mode</th>
<th>$f$ (Hz)</th>
<th>$G_E$ (MPa)</th>
<th>$\eta$</th>
<th>$k_{FCD}(f)$ (KN/\text{m})</th>
<th>$c_{FCD}(f)$ (KNs/\text{m})</th>
<th>Mode</th>
<th>$f$ (Hz)</th>
<th>$G_E$ (MPa)</th>
<th>$\eta$</th>
<th>$k_{FCD}(f)$ (KN/\text{m})</th>
<th>$c_{FCD}(f)$ (KNs/\text{m})</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.14</td>
<td>0.110</td>
<td>0.75</td>
<td>89.3</td>
<td>60.9</td>
<td>1</td>
<td>0.140</td>
<td>0.140</td>
<td>0.83</td>
<td>111.9</td>
<td>76.2</td>
</tr>
<tr>
<td>2</td>
<td>0.143</td>
<td>0.112</td>
<td>0.75</td>
<td>90.3</td>
<td>59.4</td>
<td>2</td>
<td>0.142</td>
<td>0.142</td>
<td>0.84</td>
<td>113.1</td>
<td>74.4</td>
</tr>
<tr>
<td>3</td>
<td>0.182</td>
<td>0.121</td>
<td>0.78</td>
<td>97.5</td>
<td>50.4</td>
<td>3</td>
<td>0.155</td>
<td>0.155</td>
<td>0.87</td>
<td>123.0</td>
<td>63.6</td>
</tr>
</tbody>
</table>
Fork Configuration Dampers (FCDs) for Enhanced Dynamic Performance of High-Rise Buildings

### TABLE 6-10 One-FCD-A2: SLS Across-wind Dynamic Properties for First Three Modes of Vibration

<table>
<thead>
<tr>
<th>Mode</th>
<th>$f$ (Hz)</th>
<th>Lower Bound Properties</th>
<th>FCD Properties in Shear</th>
<th>Upper Bound Properties</th>
<th>FCD Properties in Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>VE Material Properties</td>
<td>$G_E$ (MPa) $\eta$</td>
<td>$k_{FCD}(f)$</td>
<td>$c_{FCD}(f)$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>FCD Properties in Shear</td>
<td>$k_{FCD}(f)$</td>
<td>$c_{FCD}(f)$</td>
<td>$k_{FCD}(f)$</td>
</tr>
<tr>
<td>1</td>
<td>0.143</td>
<td>0.112 0.75</td>
<td>167 137</td>
<td>0.138 0.85</td>
<td>211 174</td>
</tr>
<tr>
<td>2</td>
<td>0.149</td>
<td>0.111 0.75</td>
<td>169 134</td>
<td>0.137 0.85</td>
<td>215 170</td>
</tr>
<tr>
<td>3</td>
<td>0.200</td>
<td>0.126 0.79</td>
<td>249 142</td>
<td>0.157 0.89</td>
<td>313 178</td>
</tr>
</tbody>
</table>

Tables 6-11 and 6-12 show the SLS added modal damping for Options 1 and 2, respectively. Option 1 was able to add at minimum 1% damping in the X-sway direction, more than 1.5% damping in the Y-sway direction and 3.3% damping in torsion. The average added damping for the first three modes of vibration for the upper and lower bound properties is about 2%. For all modes of vibration the natural period was increased, although less than 5% in any of the modes.

For Option 2, the FCDs added approximately 1% damping for both of the first two sway modes and 4.4% in the torsional mode without increasing the lateral periods of vibration. The average added damping was about 2% in the first three modes of vibration.

### TABLE 6-11 Option 1: SLS Across-wind Dynamic Properties

<table>
<thead>
<tr>
<th>Mode</th>
<th>Direction</th>
<th>Period (s)</th>
<th>$k_{FCD}(f)$</th>
<th>$c_{FCD}(f)$</th>
<th>$\xi_{Added}$ (%)</th>
<th>$k_{FCD}(f)$</th>
<th>$c_{FCD}(f)$</th>
<th>$\xi_{Added}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Primarily X-sway</td>
<td>7.25</td>
<td>80.6</td>
<td>45.0</td>
<td>1.18</td>
<td>7.22</td>
<td>98.7</td>
<td>53.8</td>
</tr>
<tr>
<td>2</td>
<td>Primarily Y-sway</td>
<td>7.03</td>
<td>81.4</td>
<td>43.0</td>
<td>1.69</td>
<td>6.98</td>
<td>99.8</td>
<td>52.3</td>
</tr>
<tr>
<td></td>
<td>Coupled with Torsion</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Primarily Torsion</td>
<td>5.56</td>
<td>87.4</td>
<td>36.2</td>
<td>3.32</td>
<td>5.48</td>
<td>120.5</td>
<td>63.6</td>
</tr>
<tr>
<td></td>
<td>Coupled with Y-sway</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Average 2.06 Average 2.00
CHAPTER 6: Design and Analysis of High-Rise Buildings with Fork Configuration Dampers (FCDs)

Fork Configuration Dampers (FCDs) for Enhanced Dynamic Performance of High-Rise Buildings

It was determined that for both Options 1 and 2 the human perception criteria would be met. Table 6-13 compares the SLS properties of the ETABs model for the TMD design and the FCD Options 1 and 2. Both Options 1 and 2 damped out the torsional mode of vibration significantly more than the TMD solution. Option 2, did not have an increase in period compared to the TMD solution for the torsional mode and therefore the resultant torsional velocities are substantially improved.

### TABLE 6-12 Option 2: SLS Across-wind Dynamic Properties

<table>
<thead>
<tr>
<th>Mode</th>
<th>Direction</th>
<th>Period (T) (s)</th>
<th>(k_{FCD}(f)) (KN/mm)</th>
<th>(c_{FCD}(f)) (KNs/mm)</th>
<th>(\xi_{Added}) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Primarily X-sway</td>
<td>7.02</td>
<td>167</td>
<td>137</td>
<td>1.08</td>
</tr>
<tr>
<td>2</td>
<td>Primarily Y-sway Coupled with Torsion</td>
<td>6.69</td>
<td>169</td>
<td>134</td>
<td>1.10</td>
</tr>
<tr>
<td>3</td>
<td>Primarily Torsion Coupled with Y-sway</td>
<td>5.02</td>
<td>249</td>
<td>142</td>
<td>4.38</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>2.19</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### TABLE 6-13 SLS Properties Comparison

<table>
<thead>
<tr>
<th>Modal Periods</th>
<th>Added Damping</th>
</tr>
</thead>
<tbody>
<tr>
<td>(T_1) (s)</td>
<td>(\xi_1) (%)</td>
</tr>
<tr>
<td>2-TMDs</td>
<td>6.99 6.71 5.05</td>
</tr>
<tr>
<td>Option 1</td>
<td>7.24 6.98 5.52</td>
</tr>
<tr>
<td>Option 2</td>
<td>7.01 6.70 4.98</td>
</tr>
</tbody>
</table>

An envelope of wind forces and 24 load combinations were obtained based on both Options 1 and 2 properties from the wind tunnel consultants. The maximum VE material strain of all dampers is \(\gamma_{VE1} = 64\%\) and \(\gamma_{VE2} = 57\%\) for Options 1 and 2, respectively.

To show the effects of added damping on lateral accelerations over an entire wind time-history, a wind load time-history is applied to the FE model of the original structure with an inherent damping of 1% using Option 2 with the upper bound FCD design properties. The actual wind tunnel time-history values for this building configuration were not available from the wind tunnel engineers, but a similar building base moment time-history profile was given from the wind tunnel engineers and applied as the time varying signal for a 3-Dimensional load case. The dynamic response of the structure was not included in the wind...
time-history signal, so this is sufficient to get a comparison of accelerations with added damping. A wind load time-history lasting 1000 seconds was applied and the acceleration response in the X-direction and Y-direction sway of a joint at the exterior of the floor plan are shown in Figure 6.15. As can be seen from the graphs the lateral acceleration response is reduced compared to the original structure in both directions. For this time-history the resultant maximum lateral acceleration was $a_{RMax} = 17.8\text{milli} \cdot g$ for the original structure and $a_{RMax} = 12.8\text{milli} \cdot g$ for Option 2. The FCD response of a damper on the 30th storey is shown in Figure 6.15c.

### 6.4.2.1.2 Effect of Lintel Beam Effective Stiffness on Damping

The effective stiffness of coupling beams change over the service life of the structure, as was discussed in Chapter 2. Vibration absorbers are effective over only a narrow band of natural frequency properties. Generally a vibration absorber design is optimized for a single frequency and when outside of this frequency it loses its effectiveness. The FCD adds damping for all lateral modes of vibration. To test the effects of the coupling beam effective stiffness on the dynamic properties of the Option 2 FCD solution, the coupling beam effective shear and bending properties are varied between 15% to 100% of the effective stiffness properties. The SLS properties were assumed to be 45% at all coupling beam locations for the design. The results from the FE model with varying coupling beam effective stiffnesses and the upper bound FCD design properties from Table 6-10 are shown in Figure 6.16. As the coupling beam effective stiffness reduces, the period increases for all modes of vibration investigated, as shown in Figure 6.16b. As the coupling beam effective stiffness properties reduce, the added damping increases for modes 1 and 2, however the added damping decreases for mode 3, as shown in Figure 6.16a. As the coupling beam effective stiffness increases, the added damping from the FCDs in modes 1 and 2 reduces, however the FCDs still provide a level of added damping. An interesting phenomenon is that the damping in the torsion mode (mode 3) decreases as the effective stiffness decreases, which is contrary to the conclusion made for modes 1 and 2. This could be explained by torsional shear flow continuity provided by the stiffer coupling beams.

The coupling beam effective properties change with the level of load applied to the structure, when they are uncracked it is likely that the level of the load has reached design levels and therefore not likely to cause human comfort issues, even with lower damping levels. As the load is increased and the beams crack more, the FCDs will provide more damping in the lateral directions, which is beneficial because this is when the added damping is required the most.
6.4.2.1.3 Damping Evaluation

As discussed in Section 6.4.2 the evaluation of added damping was determined using the modal damping method using ETABS damping matrix outputs. A secondary check is conducted by using the logarithmic decrement technique of the base structure and Option 2 upper bound properties with inherent damping of 1%. The free vibration is conducted by first applying a ramp loading function producing a displacement profile close to the three predominant mode shapes (X-sway, Y-sway and Torsion) for 20
seconds and then allowing the system to respond in free vibration. The displacement is measured from the center of rigidity at the 65th floor in order to remove some of the higher mode effects and to more adequately capture the response in the desired mode. Figure 6.17 shows the free vibration responses of Option 2 and of the original structure. As can be seen from the response, the original structure exhibits approximately 1% damping in all modes of vibration. Modes 1 and 2 exhibit approximately 2% total damping (1% added damping plus 1% inherent damping) for the Option 2 structure. Mode 3 exhibits approximately 5% total damping (4% added plus 1% inherent). The mode 3 response shows a bit of coupling with some of the other modes of vibration, seen in the decrement profile. The inclusion of the FCDs did not alter the period of vibration for any of the modes.

Figure 6.17 Option 2: Damping Evaluation Using Logarithmic Decrement Technique
6.4.2.1.4 Drift Criteria

The drift SLS properties are determined using the methods explained in Section 6.2.3. Based on the wind tunnel analysis the dynamic contribution to maximum strain is estimated at 60% and the static at 40%. The lower bound properties used in the drift checks are located in Tables 6-14 and 6-15 for two-FCD-As and one-FCD-A2 respectively. Since there is uncertainty in the dynamic contribution to the overall loads, a second check is done using an envelope of the assumed FCD properties varying from fully static to fully dynamic.

The resultant storey drifts for all load cases using the damping stiffness properties in the linear FEM analysis model are shown in Figure 6.18b. As shown in the figure the addition of the W-sections and FCDs in place of the concrete beams reduced the stiffness properties in the middle of the structure. As a result, the drift is increased throughout the structure. However the NBCC 2005 drift limits of 1/500 as well as the ASCE 2005 SLS drift limits of 1/400 are both still meet. When the larger FCD-A2 are used, there is only a very small increase in drifts at the locations of the FCD-A2s. These results indicate that there may be some room to further optimize the layout of the lateral load resisting system.

6.4.3 ULS Design

6.4.3.1 Wind Tunnel Study ULS 1 in 50 year Force Comparison

The wind tunnel consultants used the building SLS average modal properties of Options 1 and 2 are used to determine an estimate of the wind forces on the structure. Table 6-16 displays the base moments and base shear from the study compared to the results from the TMD model. It can be seen that the base moments and shears are comparable to the results from the TMD solution. However for Option 2, the added damping resulted in a 20% reduction in the torsional moments at the base. Option 1 had a reduction...
in the lateral base moments, but an increase in the torsional base moments. This was an interesting result considering there was more than 3% damping added torsionally. The wind tunnel consultants attributed these results to an increase in modal coupling of the first three modes of vibrations that was compounded by the change in modal frequencies that were caused by the addition of the FCD-A Option 1.

### Table 6-16 Maximum Base Moment and Base Shear Comparison

<table>
<thead>
<tr>
<th></th>
<th>Base Moments</th>
<th>Base Shear</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$M_y$</td>
<td>$M_x$</td>
<td>$M_z$</td>
<td>$F_x$</td>
</tr>
<tr>
<td></td>
<td>($10^6$ kNm)</td>
<td>($10^6$ kNm)</td>
<td>($10^3$ kNm)</td>
<td>($10^3$ kNm)</td>
</tr>
<tr>
<td>2-TMDs</td>
<td>2.81</td>
<td>2.01</td>
<td>101</td>
<td>17.2</td>
</tr>
<tr>
<td>Option 1</td>
<td>2.81</td>
<td>2.01</td>
<td>111</td>
<td>16.9</td>
</tr>
<tr>
<td>Option 2</td>
<td>2.76</td>
<td>2.00</td>
<td>80</td>
<td>17.2</td>
</tr>
</tbody>
</table>

**6.4.3.2 ULS Adjacent Member Design**

The properties are obtained assuming the wind load is 60% dynamic, 40% static at the natural period of vibration of the structure with the method proposed in Section 6.2.3. The properties used in the analysis are located in Tables 6-17 and 6-18. The shear forces in the adjacent left concrete coupling beam locations along the same elevation are located in Figure 6.19. As seen in the figure the factored design shear forces do increase slightly with the FCD Option 1 compared to the two-TMD option, however the strength limits of all the members are still well within the concrete beam strength capacity. For Option 2 it can be seen that there is a reduction of the factored design shear forces in the left line of coupling beams. Figure 6.20
displays the factored design shear forces of the W-sections for Option 1 and concrete beams for Option 2 above and below the FCDs. As can be seen in the figure, the forces are redistributed when the FCDs are added, however Option 2 has an increase in coupling beam and FCD forces. There is also a spike in the forces above the FCD in the concrete coupling beams. The stiffness parameters would probably be reduced in the final detailed design stage to achieve a better stiffness distribution. In all cases, the strength criteria for all members were satisfied.

**Figure 6.19 Governing Resultant Adjacent Member Design Forces (Left Line Coupling Location)**

**Figure 6.20 Governing Member Design Forces In-line with FCD (Right Line Coupling Location)**
CHAPTER 6: Design and Analysis of High-Rise Buildings with Fork Configuration Dampers (FCDs)

6.4.3.3 ULS Connection Design

The ULS FCD connection design forces are assumed to be governed by a fully dynamic loading case, at the natural frequency of the building using the method described in Section 6.2.3. Tables 6-19 and 6-20 display the VE material properties assumed in the connection design. Using the maximum damper displacement from the linear elastic FE model, \( u_{\text{FCD}0} \), the maximum factored FCD connection force is estimated:

\[
F_{f\text{FCD}} = 1.4\left( k_{\text{FCD}} u_{\text{FCD}0} / \cos \theta_{\text{FCD}} \right)
\]  

(6.15)

### TABLE 6-17 Two-FCD-A: ULS Adjacent Member Design

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Design Bound</th>
<th>VE Material Properties</th>
<th>FCD Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( k_{\text{VES}} ) ( k_{\text{VE}} ) ( c_{\text{VE}} )</td>
<td>( k_{\text{FCDS}} ) ( k_{\text{FCD}} ) ( c_{\text{FCD}} ) ( k_{\text{FCDEff}} )</td>
</tr>
<tr>
<td>Along-wind ULS</td>
<td>Lower Bound</td>
<td>14.4 74.5 54.0</td>
<td>14.0 68.0 46.3 46.4</td>
</tr>
</tbody>
</table>

### TABLE 6-18 One-FCD-A2 ULS Adjacent Member Design

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Design Bound</th>
<th>VE Material Properties</th>
<th>FCD Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( k_{\text{VES}} ) ( k_{\text{VE}} ) ( c_{\text{VE}} )</td>
<td>( k_{\text{FCDS}} ) ( k_{\text{FCD}} ) ( c_{\text{FCD}} ) ( k_{\text{FCDEff}} )</td>
</tr>
<tr>
<td>Along-wind ULS</td>
<td>Lower Bound</td>
<td>33.3 207 149.9</td>
<td>33.2 196.9 156.7 144.4</td>
</tr>
</tbody>
</table>

### TABLE 6-19 Two-FCD-A: ULS Connection Design

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Design Bound</th>
<th>VE Material Properties</th>
<th>FCD Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( G_E ) ( \eta ) ( k_{\text{VE}} ) ( c_{\text{VE}} )</td>
<td>( k_{\text{FCD}} ) ( c_{\text{FCD}} )</td>
</tr>
<tr>
<td>Across-wind ULS</td>
<td>Upper Bound</td>
<td>0.139 0.842 125 121</td>
<td>111 75.9</td>
</tr>
</tbody>
</table>

### TABLE 6-20 One-FCD-A2: ULS Connection Design

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Design Bound</th>
<th>VE Material Properties</th>
<th>FCD Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( G_E ) ( \eta ) ( k_{\text{VE}} ) ( c_{\text{VE}} )</td>
<td>( k_{\text{FCD}} ) ( c_{\text{FCD}} )</td>
</tr>
<tr>
<td>Across-wind ULS</td>
<td>Upper Bound</td>
<td>0.139 0.842 125 121</td>
<td>111 75.9</td>
</tr>
</tbody>
</table>

Figure 6.20 displays the FCDs factored shear force; the FCD ultimate strength for all members is satisfied.
6.4.4 Earthquake Time-History

To demonstrate the equivalent modelling application to earthquake time-histories using commercial software, the tower was subjected to the Landers 1992 time-history scaled to the Toronto response spectrum. Considering high-rise buildings are not typically designed to have more than nominal ductility in Toronto the model was completely linear elastic with the dampers modelled using the equivalent VE properties in the first mode of vibration. The response of the top storey displacement of the damped and undamped models are shown in Figure 6.21a and the force-displacement hysteresis of an FCD on the thirtieth floor is shown in Figure 6.21b. As can be seen the top storey displacement of the structure with FCDs is reduced slightly compared to the original structure. Also, the hysteresis shows higher mode force spikes in the FCD response, discussed in Chapter 5.

![Figure 6.21 Landers 1992, Scaled to Match Toronto Response Spectrum](image)

6.4.5 Conclusions from Case Study

The 85-storey structure was designed using two arrangements of FCDs. Option 1 consisted of two-FCD-As replacing 122 RC coupling beams and Option 2 consisted of a larger damper, FCD-A2, replacing 104 RC coupling beams. The results from the study were compared to results from a design including two-TMD design at the top storey of the structure. Both FCD solutions provided added damping to the lateral modes of vibrations. In Option 1, the natural periods of vibration were altered by less than 5%, while Option 2 did not alter the periods of vibration. The design criteria for all the SLS (human perception criteria and drift limits) and ULS (adjacent member design and FCD connection design) were satisfied. The base moments and forces were comparable to the two-TMD solution as well as the other design parameters. No attempt was made to alter the structural configurations or to optimize the solution through removing walls or changing wall thicknesses. Optimization of the structural configuration could result in a result in a reduction of construction materials.
CHAPTER 6: Design and Analysis of High-Rise Buildings with Fork Configuration Dampers (FCDs)

The increase of the usable space at the top of the building was seen as a large benefit to the owner, because the selling price per square foot at that storey was more than twice as much as the average price per square foot of the building. Also, in comparison to the two-TMDs, the FCDs would require little to no maintenance, while the TMDs would need to be inspected and maintained throughout the life of the building. Also, the two-TMD design was optimized for the SLS 1 in 10 year load, based on the assumed cracking properties at that load level. No information was given for the performance of the TMDs if other cracking properties or load levels occur over the life of the building. Monitoring and subsequent tuning could only verify the design effectiveness for the actual in-situ building properties, meanwhile the FCD system will provide damping for all deformation levels and all lateral modes of vibration throughout the life of the building.

6.4.5.1 Dynamic Performance Comparison

To demonstrate the dynamic performance of the FCD Option 2 compared to the original structure and the two-TMD design, the SLS and ULS responses are shown in Figures 6.22 and 6.23. Since the natural period of vibration of Option 2 is not much different from the original structure period it can be compared directly. As can be seen from Figure 6.22 the lateral acceleration response of the structure is relatively the same as the two-TMD solution, however the Option 2 performance will be slightly better, because of the added damping in mode 3 (torsional mode) which contributes slightly to the response in the lateral direction. The acceleration reduction of the original structure is approximately 35% when 1% inherent damping is assumed and about 25% when 2% inherent damping is assumed. The torsional velocities are significantly improved compared to both the original structure and the two-TMD solution, because of the large amount of torsional added damping. The torsional velocities improve by over 50% when 1% inherent damping is assumed and 35% when 2% inherent damping is assumed.

Figure 6.23 shows the comparison of base moments as a function of damping. The reduction in lateral base moments for Option 2 is comparable to the performance of the two-TMD solution, representing about a 20% reduction in lateral base moments when 1% inherent damping is assumed and about a 15% reduction in the lateral base moments when 2% inherent damping is assumed. The torsional base moments are reduced significantly compared to the original structure and the two-TMD solution, due to the significant added damping. When 1% inherent damping is assumed, the torsional base moments are reduced by approximately 35% and when 2% inherent damping is assumed, the torsional base moments are reduced by about 25%.
CHAPTER 6: Design and Analysis of High-Rise Buildings with Fork Configuration Dampers (FCDs)

FORK CONFIGURATION DAMPERS (FCDS) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS

Figure 6.22 SLS 1 in 10 year Dynamic Response as a Function of Damping

Figure 6.23 ULS 1 in 50 Base Moments as a Function of Damping
CHAPTER 7: SUMMARY AND CONCLUSIONS

7.1 OVERVIEW OF THESIS

A new, distributed damping system for reinforced concrete coupled wall buildings, the Fork Configuration Damper (FCD), has been developed and experimentally validated in this thesis. The FCDs are introduced in lieu of concrete coupling beams connecting two walls, where large shear deformations occur during lateral dynamic vibrations. The dampers utilize viscoelastic (VE) material acting in shear, where the hysteretic response of the VE material to dynamic excitations is a combination of linear and viscous components. The FCD consists of multiple layers of VE material bonded and sandwiched between multiple steel plates. The VE material alternates between layers of steel plates with each consecutive steel plate extending out to the opposite sides and bolted together in a slip-critical connection with built-up I-sections and filler plates. Under lateral vibrations the walls deflect laterally which cause large rotations about the wall centerlines. These large rotations cause relative shear displacements at both ends of the FCD producing large shear deformations in the VE material. Through this action, the FCDs add distributed damping throughout the high-rise building which results in supplemental damping to the lateral modes of vibration. This added damping reduces the dynamic response of the structure to wind vibrations and moderate earthquakes where the response is primarily linear. For extreme loads (such as large earthquakes) a built-in structural force limiting “fuse” mechanism was developed. The structural “fuse” is configured in series with the VE material and steel plate layers and is activated when a specified force is achieved during an extreme loading event such as a rare earthquake. The “fuse” is capacity designed to ensure the VE material is not damaged and that all structural damage occurs in the “fuse” itself which is designed to have a ductile response, thus ensuring unwanted or unexpected brittle failure mechanisms of the system can not occur.

An experimental program was conducted to validate the new FCD system and concept. Two series of tests were conducted, with 11 specimens (5 small-scale and 6 full-scale) provided in-kind by Nippon Steel Engineering Co. (NSEC). The first set of tests were on small-scale VE damper specimens with two different VE layer thicknesses of 5 and 10 mm conducted at the University of Toronto Structural Testing Facilities (UofT). The test set was used to characterize and evaluate the VE material behaviour under
realistic loading conditions expected in high-rise buildings. These tests confirmed that the VE material hysteretic behaviour was stable and predictable under representative high-rise building loading conditions. VE material properties characterized by the small-scale tests compared well to design properties suggested by the damper manufacturer. Under representative wind storm loading the damper VE material did not have substantial self-heating effects.

The second set of tests was conducted on two full-scale FCD samples at École Polytechnique in Montreal (EPM). The first specimen was designed for a building located in downtown Toronto and the second damper was designed for a high-rise building in a seismic region such as Vancouver. There were two different connection types tested: i) FCD-A, with built-up I-sections welded to an end-plate cast into the concrete wall, to be used in designs where low to moderate ductility is required, and ii) FCD-B, with built-up Reduced Beam Section (RBS) “fuses” welded to an end-plate which was cast into the concrete wall, to be used in designs where large ductility is required.

The test setup consisted of multiple precast concrete walls post-tensioned together and tested using a racking configuration to simulate realistic FCD behaviour. The full-scale tests were used to validate the overall system performance based on the kinematic behavior of coupled walls, wall anchorage and VE material behavior during actual design level loads. Both samples were subject to harmonic characterization tests for a range of frequencies and strains to characterize the FCD and VE performance. Next, sets of wind and earthquake time-histories of analytically determined FCD response in high-rise buildings were conducted. The results from the full-scale harmonic characterization and the time-histories showed stable and predictable behaviour of the VE material. Finally, the FCDs were cycled to failure under quasi-static or dynamics loads, to establish the ultimate behaviour of the FCDs. The ultimate level tests showed a predictable and reliable response at large load levels. The dynamic tests showed that the FCD shear force levels were limited and VE material showed only superficial tearing up to large non-linear deformations and the concrete walls remained essentially uncracked for all tests.

The VE harmonic material behaviour was validated using results from the small-scale VE material tests using a Kelvin-Voigt model (referred to as model “KVM”) and a generalized Maxwell model (referred to as model “GMM4”), which is able to capture self-heating, strain and frequency-dependent VE material properties. A temperature correction factor was then introduced to the generalized Maxwell model (referred to as model “GMM4TC”) to attempt to account for heat conduction and convection of the VE material and steel plates that was observed for long duration tests. The GMM4TC model captured the behaviour well over the duration of the tests, because incorporated in the model is a method to calculate the temperature change. The KVM damping and stiffness coefficients are based on VE material properties at a constant strain amplitude, frequency and temperature. The KVM was able to model the VE material response well when all of these conditions could be approximated.
After the VE material validation, modelling techniques developed by Kasai et. al (2006) for VE dampers in brace configurations were applied to model the shear behaviour of the FCD. The techniques take into account the steel damper assembly connection stiffness and the VE material stiffness producing an equivalent VE model of the FCD (model 1). Next a Kelvin-Voigt model in series with a spring (model 2) was used to represent the FCD behaviour. Finally the generalized Maxwell model with a spring in series (model 3) was used to model the VE material temperature, strain and frequency dependency of the FCD. The models were validated against the full-scale test results. The proposed models for the FCD managed to capture the overall shear behaviour of the FCD well. Model 1 (equivalent VE model) and Model 2 (spring in series with the KVM) managed to capture the test results well when the VE material temperature could be approximated for the harmonic characterization and wind test results. For wind loading a bounded analysis approach could be used effectively. However, when the excitation was dependent on higher modes of vibration, such as earthquakes, the response was not well captured because of the multiple mode response. Model 3 (spring in series with the GMM) captured the response well for all types of loading and was able to approximate the temperature over the full duration of the test. The higher mode effects for the earthquake tests were captured well using Model 3.

Finally a design methodology was proposed for high-rise buildings for both wind and earthquake loading. The methodology and tools developed were then used to conduct two alternative designs of an 85-storey coupled wall building incorporating the FCD system, located in downtown Toronto. The inclusion of FCDs, added damping to all lateral modes of vibration, reducing the dynamic response of the structure for all loading conditions over a range of building natural periods.

### 7.2 COMPARISON OF FCD SYSTEM TO CURRENT VIBRATION ABSORBERS

A comparison of the performance between structures using the FCD system and structures incorporating a vibration absorber is summarized in Table 7-1. This comparison is carried out both for structural and architectural or owner considerations. The structural considerations represent comparisons on the performance of both systems and the structural design criteria. The architectural or owner considerations are economic in nature and represent overall project design criteria.
### TABLE 7-1 Comparison of Vibration Absorber and FCD System

<table>
<thead>
<tr>
<th></th>
<th><strong>Vibration Absorber</strong></th>
<th><strong>FCD System</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Structural Considerations</strong></td>
<td>• Designed and optimized to add effective damping in one or two modes of vibration and therefore only useful for vibrations that cause the building to respond in those modes. i.e. they are not useful for reducing earthquake vibrations etc.</td>
<td>• Adds distributed viscous damping to all lateral modes of vibration. Therefore FCDs are useful for mitigating the response to any lateral dynamic loads including both earthquake and wind vibrations.</td>
</tr>
<tr>
<td></td>
<td>• During extreme events, such as earthquakes, yielding is expected in structural members. Structural members must be designed to resist the effects induced by the additional mass at the top storey. When this occurs there is a de-tuning effect and the vibration absorbers become less effective.</td>
<td>• Added damping reduces the dynamic response to large earthquakes and a capacity design procedure is used to protect against non-ductile actions of the FCD. Change of structural or VE material properties have little effect on overall effectiveness on added damping.</td>
</tr>
<tr>
<td></td>
<td>• The vibration absorber can be designed after the lateral load resisting system has been designed, but a gravity system needs to support the added weight of the vibration absorber system.</td>
<td>• The FCD system can be designed to fit within a current lateral load resisting system, however it is best included in the design at the beginning in order to optimize the lateral load resisting system.</td>
</tr>
<tr>
<td></td>
<td>• It may not be tuned adequately for unexpected loading conditions, due to the variability of dynamic properties and therefore can not be used reliably to reduce the size of structural members. For example, if vibration absorber is tuned for the structural properties of the building for 1 in 10 year wind loads, but they may not be tuned adequately for the structural properties for more extreme load conditions when the added damping is required.</td>
<td>• Reduces dynamic inertia loads for all loading conditions if FCDs are placed and designed properly. This can be advantageous for reducing the size of the RC structure.</td>
</tr>
<tr>
<td></td>
<td>• Must design elements globally to resist added weight and locally to resist local loads due to the large moving mass.</td>
<td>• Lower static stiffness increases deflections caused by static portion of wind loading in the along-wind direction.</td>
</tr>
<tr>
<td><strong>Architectural or Owner Considerations</strong></td>
<td>• Is an additional cost to design and install.</td>
<td>• Is an added cost over the coupling beams, however may be constructed faster on site.</td>
</tr>
<tr>
<td></td>
<td>• Takes up significant space of valuable real-estate.</td>
<td>• Non-obtrusive and non-invasive.</td>
</tr>
<tr>
<td></td>
<td>• Must be monitored to validate adequate performance over the life of the structure. Also, must be maintained and inspected to ensure performance.</td>
<td>• Minimal inspection required to ensure adequate performance. Additional studies being carried out to further confirm long-term VE material properties.</td>
</tr>
<tr>
<td></td>
<td>• More building damage expected relative to FCD system if a large earthquake occurs, because of more nonlinear behaviour expected in structural members.</td>
<td>• By delaying the onset of damage to the structure during extreme earthquake events could result in less downtime compared to vibration absorbers.</td>
</tr>
<tr>
<td></td>
<td>• Due to the variability in the dynamic properties and the possibility that the TMD may de-tune, TMDs are typically not used to reduce structural member sizes.</td>
<td>• Can potentially reduce structural member sizes by reducing the inertia loads. This would allow more sellable space, more flexible unit division. Also, this would reduce the overall material cost, labour and construction time.</td>
</tr>
<tr>
<td></td>
<td>• One design solution.</td>
<td>• Flexible designs possible, i.e. could be used in locations preferable to architects.</td>
</tr>
<tr>
<td></td>
<td>• Constructed towards end of project and needs to be re-calibrated during the life of the structure.</td>
<td>• Modular construction.</td>
</tr>
<tr>
<td></td>
<td>• Has been used in the past.</td>
<td>• Has not yet been used.</td>
</tr>
</tbody>
</table>
7.3 POTENTIAL NEW DESIGN PHILOSOPHIES AND POTENTIAL ADDITIONAL BENEFITS

The FCD technology is sufficiently different from the current best practice or “state-of-the-art” design. Current design strategy are to stiffen the structure significantly and if the structure is still dynamically sensitive, a vibration absorber is added. The full benefits or drawbacks of the FCD system will be ultimately best determined by assessment from multiple parties: i) building developers ii) architects, iii) structural engineers and iv) wind tunnel engineers. Also, high-rise buildings are typically non-standard with different structural layouts, unit division, geographic location, governing load conditions and building cladding shape therefore, every building typically has new design requirements. For these reasons, using the FCD allows for more creative solutions to the dynamic problems faced by designers. This could in fact, be the biggest benefit for the applicability and use of the technology.

It is possible that the full benefits of the FCD system have not yet fully been identified. As an example of this, since typical design practice consists of stiffening the structure so as to avoid vibration problems, it may be possible to work in conjunction with the architects, contractors and structural engineers to optimize a solution with FCDs for structures ranging from 40 to 60 stories tall. These structures typically would not require a vibration absorber system to mitigate vibration perception problems, and are typically made very stiff to satisfy the SLS human perception design criteria. It could be possible to design a universal RC core equipped with FCDs, that could add significant damping and optimize the use of concrete in the walls. This could change the entire design process of iteration with the architects, contractors, structural engineers and wind tunnel engineers and significantly streamline the design.
7.4 RECOMMENDATIONS FOR FURTHER RESEARCH

A number of research studies can be conducted to expand on the FCD research and development that has been presented in this thesis. A number of recommendations for further studies are described below.

The additional FCD details that were discussed in Chapter 3 could be investigated and tested, including:

- Alternative FCD connection details including: cast-in-place I-section details, bolted or post-tensioning details, splice plate details and weld connection details.
- Alternative “fuse” mechanisms including: shear force limiting links, force limiting anchors and friction force limiting links.
- Replaceable connections that could be inspected and replaced after an extreme earthquake event, to reducing significant repair costs.
- Alternative configurations in the lateral load resisting system (e.g. outrigger or tube systems).
- Localized effects of the slab. This would determine the effects of coupling of stiffness between the slab and the FCD or determine if the slab requires special connection details to allow free movement.

The effect of the FCDs’ relatively low static stiffness on the overall building response to wind should be studied further. This could cause stiffness irregularities along the lintel beam perforations up the height of the structure. In particular, this would likely be at storeys directly above or below the FCDs if the lintel beams are relatively stiff. As discussed in Chapter 2, when faced with stiffness irregularities of adjacent lintel beams, structural engineers use an iterative procedure to redistribute loads using beam cracking parameters in highly stressed locations. If the beams directly above the FCD are highly stressed, the iterative cracking procedure could be used redistribute the concrete member stiffnesses. Another method to reduce the stiffness irregularities would be to use less stiff structural sections adjacent to the FCDs. These could include shallower concrete beam sections or structural steel sections, which was demonstrated in Chapter 5. Alternatively, the stiffness and damping of the FCD could be increased by adding more layers of VE material and steel plates or by increasing the VE area per layer. The size of steel sections in the assembly connection could also be increased which would increase the effective stiffness and damping of the FCD. The FCDs’ relatively low static stiffness is considered a property that can be accommodated by design options by using a combination of some of the methods described above. In the 85-storey case study design a FCD drift envelope compared the drift profile of the FCDs ranging from fully static VE properties to fully dynamic VE properties. In the case of the fully static properties the interstorey drifts are increased, although still below the drift criteria for both design options presented. The drift envelope and adjacent member force envelope are effective ways to visualize the problem and provide the upper and
lower design bounds. Current design (discussed in Chapter 2) seems to be heading towards relaxing the drift limits for service level wind vibrations. This is especially the case when the building deformations are cantilever type bending deformations, that are not necessarily associated with damage that is usually linked to interstorey deformations.

Currently, the design process relies on wind tunnel studies to determine the full envelope of design wind loads. The wind tunnel consultants need to be asked specifically to separate out the dynamic and static loads to better understand the effects of this relatively lower static stiffness in the structural response.

Currently, typical design studies are conducted on completely bare lateral load resisting structural models, i.e. the only elements modelled are the lateral load resisting structural elements, such as walls, beams, columns and slabs modelled as rigid diaphragms. Some structural elements that actually add lateral stiffness to the overall building, but are typically not modelled, include the stairs and stairwells and slabs in shear and bending. In additional to these structural elements, are architectural elements that add stiffness to the overall lateral load resisting system including partitions, drywall and cladding. These adjacent elements would influence the amount of added damping to the overall structure, because of their stiffness contribution, especially if they are directly adjacent to the FCDs. Further study on whether these effects are significant or if they impact the overall added damping should be carried out. Also, studies on other details that could reduce these effects could be developed.

Further optimization techniques for the FCD system could be investigated including the possibility of developing specialized software to determine the optimal FCD locations, the required number of FCDs and the adjacent structural members. It would be useful if the software was 3-Dimensional and based on methods used for current high-rise design.

Development of a coupled wall lateral load resisting system that uses FCDs in conjunction with ductile steel or ductile concrete elements distributed up the height of the structure could be beneficial design. This could offer a robust design solution that has added damping for wind vibrations when the ductile steel or concrete elements behave linearly and when an extreme seismic event occurs the members are expected to yield with sufficient distributed ductility coming from the ductile steel or ductile concrete elements. Research to further investigate this is currently underway at the University of Toronto.

Another study could include the reduction of concrete wall thickness over the building height, due to the reduction of inertia loads that are achieved as a result of the added damping. This could result in a decrease in overall building material, labour and construction costs over the height of the building.
The applicability of using other VE materials in the FCD configuration should also be investigated. These materials could have different stiffness and damping properties that could be more beneficial and more effective than those thus far inspected.

Studies on the long-term aging effects and the fatigue life of the VE ISD-111H material would be very beneficial for implementation in a real structure to confirm that this VE material has similar stable behaviour as other ISD materials that have been investigated over a number of decades. Some fatigue tests are currently being conducted in the laboratories at UofT.

Methods for the inspection of the FCDs post earthquakes or wind-storms for structural integrity could also be investigated.

A more thorough cost analysis of the FCD system would be very useful for potential users of this technology. This would emphasize two design aspects: i) the cost effectiveness of the current FCD connection details, and ii) an overall cost-benefit analysis of a structure incorporating FCDs. The global cost-benefit analysis of a structure with distributed FCD elements would help the potential use of the FCD design in real high-rise buildings.

Lastly, considering this technology for the retrofit of existing high-rise buildings could be useful to investigate and test. Since the FCDs are modular units it may be possible to design an anchorage connection where the FCD could be used to replace deficient concrete or steel link beams in structures. There also could be the possibility of replacing beams that are damaged during earthquakes with FCDs when repairing the structure.
REFERENCES


ACI (2004). “Building code requirements for structural concrete (ACI 318-05) and commentary (ACI 318R-05)”, American Concrete Institute, Farmington Hills, MI.


ASCE (2006). “ASCE/SEI 7-05 minimum design loads for buildings and other structures”, American Society of Civil Engineers, Reston, VA.


AIJ (2004). “Guidelines for the evaluation of habitability to building vibration”, Architectural Institute of Japan, Tokyo, JP.


References


FORK CONFIGURATION DAMPERS (FCDs) FOR ENHANCED DYNAMIC PERFORMANCE OF HIGH-RISE BUILDINGS


Referenced materials or sources for Fork Configuration Dampers (FCDs) for Enhanced Dynamic Performance of High-Rise Buildings.
References


References


NIST (2005). “NIST NCSTAR 1-1, federal building and fire safety investigation of the world trade center disaster - design, construction, and maintenance of structural and life safety systems”, National Institute of Standards and Technology, Gaithersburg, MD.


Rosman, R. (1964). “Approximate analysis of shear walls subject to lateral loads”, Journal of the American Concrete Institute, 61, 716-733.


References


