Evaluation of Fluid Dampers for Seismic Energy Dissipation of Woodframe Structures

Michael D. Symans
William F. Cofer
Ying Du
Kenneth Fridley

2002
The CUREE-Caltech Woodframe Project is funded by the Federal Emergency Management Agency (FEMA) through a Hazard Mitigation Grant Program award administered by the California Governor’s Office of Emergency Services (OES) and is supported by non-Federal sources from industry, academia, and state and local government. California Institute of Technology (Caltech) is the prime contractor to OES. The Consortium of Universities for Research in Earthquake Engineering (CUREE) organizes and carries out under subcontract to Caltech the tasks involving other universities, practicing engineers, and industry.

Disclaimer

The information in this publication is presented as a public service by California Institute of Technology and the Consortium of Universities for Research in Earthquake Engineering. No liability for the accuracy or adequacy of this information is assumed by them, nor by the Federal Emergency Management Agency and the California Governor’s Office of Emergency Services, which provide funding for this project.
Evaluation of Fluid Dampers for Seismic Energy Dissipation of Woodframe Structures

Michael D. Symans
Department of Civil and Environmental Engineering
Rensselaer Polytechnic Institute

William F. Cofer
Ying Du
Department of Civil and Environmental Engineering
Washington State University

Kenneth J. Fridley
Howard R. Hughes College of Engineering
University of Nevada, Las Vegas

2002
Published by
Consortium of Universities for Research in Earthquake Engineering (CUREE)
1301 S. 46th Street - Richmond, CA 94804-4600
www.curee.org (CUREE Worldwide Website)
Preface

The CUREE-Caltech Woodframe Project originated in the need for a combined research and implementation project to improve the seismic performance of woodframe buildings, a need which was brought to light by the January 17, 1994 Northridge, California Earthquake in the Los Angeles metropolitan region. Damage to woodframe construction predominated in all three basic categories of earthquake loss in that disaster:

- **Casualties:** 24 of the 25 fatalities in the Northridge Earthquake that were caused by building damage occurred in woodframe buildings (1);
- **Property Loss:** Half or more of the $40 billion in property damage was due to damage to woodframe construction (2);
- **Functionality:** 48,000 housing units, almost all of them in woodframe buildings, were rendered uninhabitable by the earthquake (3).

Woodframe construction represents one of society’s largest investments in the built environment, and the common woodframe house is usually an individual’s largest single asset. In California, 99% of all residences are of woodframe construction, and even considering occupancies other than residential, such as commercial and industrial uses, 96% of all buildings in Los Angeles County are built of wood. In other regions of the country, woodframe construction is still extremely prevalent, constituting, for example, 89% of all buildings in Memphis, Tennessee and 87% in Wichita, Kansas, with "the general range of the fraction of wood structures to total structures...between 80% and 90% in all regions of the US…." (4).

Funding for the Woodframe Project is provided primarily by the Federal Emergency Management Agency (FEMA) under the Stafford Act (Public Law 93-288). The federal funding comes to the project through a California Governor’s Office of Emergency Services (OES) Hazard Mitigation Grant Program award to the California Institute of Technology (Caltech). The Project Manager is Professor John Hall of Caltech. The Consortium of Universities for Research in Earthquake Engineering (CUREE), as subcontractor to Caltech, with Robert Reitherman as Project Director, manages the subcontracted work to various universities, along with the work of consulting engineers, government agencies, trade groups, and others. CUREE is a non-profit corporation devoted to the advancement of earthquake engineering research, education, and implementation. Cost-sharing contributions to the Project come from a large number of practicing engineers, universities, companies, local and state agencies, and others.

The project has five main Elements, which together with a management element are designed to make the engineering of woodframe buildings more scientific and their construction technology more efficient. The project’s Elements and their managers are:

1. **Testing and Analysis:** Prof. André Filiatrault, University of California, San Diego, Manager; Prof. Frieder Seible and Prof. Chia-Ming Uang, Assistant Managers
2. **Field Investigations:** Prof. G. G. Schierle, University of Southern California, Manager
3. **Building Codes and Standards:** Kelly Cobeen, GFDS Engineers, Manager; John Coil and James Russell, Assistant Managers
4. **Economic Aspects:** Tom Tobin, Tobin Associates, Manager
5. **Education and Outreach:** Jill Andrews, Southern California Earthquake Center, Manager
The Testing and Analysis Element of the CUREE-Caltech Woodframe Project consists of 23 different investigations carried out by 16 different organizations (13 universities, three consulting engineering firms). This tabulation includes an independent but closely coordinated project conducted at the University of British Columbia under separate funding than that which the Federal Emergency Management Agency (FEMA) has provided to the Woodframe Project. Approximately half the total $6.9 million budget of the CUREE-Caltech Woodframe Project is devoted to its Testing and Analysis tasks, which is the primary source of new knowledge developed in the Project.

Woodframe Project Testing and Analysis Investigations

<table>
<thead>
<tr>
<th>Task #</th>
<th>Investigator</th>
<th>Topic</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td><strong>Project-Wide Topics and System-level Experiments</strong></td>
</tr>
</tbody>
</table>
| 1.1.1  | André Filiatrault, UC San Diego  
Kelly Cobeen, GFDS Engineers | Two-Story House (testing, analysis)  
Two-Story House (design) |       |
| 1.1.2  | Khalid Mosalam, Stephen Mahin, UC Berkeley  
Bret Lizundia, Rutherford & Chekene | Three-Story Apt. Building (testing, analysis)  
Three-Story Apt. Building (design) |       |
| 1.1.3  | Frank Lam et al., U. of British Columbia | Multiple Houses (independent project funded separately in Canada with liaison to CUREE-Caltech Project) |       |
| 1.2    | Bryan Folz, UC San Diego | International Benchmark (analysis contest) |       |
| 1.3.1  | Chia-Ming Uang, UC San Diego | Rate of Loading and Loading Protocol Effects |       |
| 1.3.2  | Helmut Krawinkler, Stanford University | Testing Protocol |       |
| 1.3.3  | James Beck, Caltech | Dynamic Characteristics |       |
|        |              | **Component-Level Investigations** |       |
| 1.4.1.1| James Mahaney; Wiss, Janney, Elstner Assoc. | Anchorage (in-plane wall loads) |       |
| 1.4.1.2| Yan Xiao, University of Southern California | Anchorage(hillside house diaphragm tie-back) |       |
| 1.4.2  | James Dolan, Virginia Polytechnic Institute | Diaphragms |       |
| 1.4.3  | Rob Chai, UC Davis | Cripple Walls |       |
| 1.4.4.4| Gerard Pardoen, UC Irvine | Shearwalls |       |
| 1.4.6  | Kurt McMullen, San Jose State University | Wall Finish Materials (lab testing) |       |
| 1.4.6  | Gregory Deierlein, Stanford University | Wall Finish Materials (analysis) |       |
| 1.4.7  | Michael Symans, Washington State University | Energy-Dissipating Fluid Dampers |       |
| 1.4.8.1| Fernando Fonseca, Brigham Young University | Nail and Screw Fastener Connections |       |
| 1.4.8.2| Kenneth Fridley, Washington State University | Inter-Story Shear Transfer Connections |       |
| 1.4.8.3| Gerard Pardoen, UC Irvine | Shearwall-Diaphragm Connections |       |
|        |              | **Analytical Investigations** |       |
| 1.5.1  | Bryan Folz, UC San Diego | Analysis Software Development |       |
| 1.5.2  | Helmut Krawinkler, Stanford University | Demand Aspects |       |
| 1.5.3  | David Rosowsky, Oregon State University | Reliability of Shearwalls |       |
Not shown in the tabulation is the essential task of managing this element of the Project to keep the numerous investigations on track and to integrate the results. The lead management role for the Testing and Analysis Element has been carried out by Professor André Filiatrault, along with Professor Chia-Ming Uang and Professor Frieder Seible, of the Department of Structural Engineering at the University of California at San Diego.

The type of construction that is the subject of the investigation reported in this document is typical “two-by-four” frame construction as developed and commonly built in the United States. (Outside the scope of this Project are the many kinds of construction in which there are one or more timber components, but which cannot be described as having a timber structural system, e.g., the roof of a typical concrete tilt-up building). In contrast to steel, masonry, and concrete construction, woodframe construction is much more commonly built under conventional (i.e., non-engineered) building code provisions. Also notable is the fact that even in the case of engineered wood buildings, structural engineering analysis and design procedures, as well as building code requirements, are more based on traditional practice and experience than on precise methods founded on a well-established engineering rationale. Dangerous damage to US woodframe construction has been rare, but there is still considerable room for improvement. To increase the effectiveness of earthquake-resistant design and construction with regard to woodframe construction, two primary aims of the Project are:

1. Make the design and analysis more scientific, i.e., more directly founded on experimentally and theoretically validated engineering methods and more precise in the resulting quantitative results.

2. Make the construction more efficient, i.e., reduce construction or other costs where possible, increasing seismic performance while respecting the practical aspects associated with this type of construction and its associated decentralized building construction industry.

The initial planning for the Testing and Analysis tasks evolved from a workshop that was primarily devoted to obtaining input from practitioners (engineers, building code officials, architects, builders) concerning questions to which they need answers if they are to implement practical ways of reducing earthquake losses in their work. (Frieder Seible, André Filiatrault, and Chia-Ming Uang, Proceedings of the Invitational Workshop on Seismic Testing, Analysis and Design of Woodframe Construction, CUREE Publication No. W-01, 1999.) As the Testing and Analysis tasks reported in this CUREE report series were undertaken, each was assigned a designated role in providing results that would support the development of improved codes and standards, engineering procedures, or construction practices, thus completing the circle back to practitioners. The other elements of the Project essential to that overall process are briefly described below.

To readers unfamiliar with structural engineering research based on laboratory work, the term “testing” may have a too narrow a connotation. Only in limited cases did investigations carried out in this Project “put to the test” a particular code provision or construction feature to see if it “passed the test.” That narrow usage of “testing” is more applicable to the certification of specific models and brands of products to declare their acceptability under a particular product standard. In this Project, more commonly the experimentation produced a range of results that are used to calibrate analytical models, so that relatively expensive laboratory research can be applicable to a wider array of conditions than the single example that was subjected to simulated earthquake loading. To a non-engineering bystander, a “failure” or “unacceptable damage” in a specimen is in fact an instance of successful experimentation if it provides a valid set of data that builds up the basis for quantitatively predicting how wood components and systems of a wide variety will perform under real earthquakes. Experimentation has also been conducted to improve the starting point for this kind of research: To better define what specific kinds of simulation in the laboratory best represent the real conditions of actual buildings subjected to earthquakes, and to develop protocols that ensure data are produced that serve the analytical needs of researchers and design engineers.
Notes


Acknowledgments

This research was carried out under contract to Consortium of Universities for Research in Earthquake Engineering (CUREE) as part of the CUREE-Caltech Woodframe Project (“Earthquake Hazard Mitigation of Woodframe Construction”), under a grant administered by the California Office of Emergency Services and funded by the Federal Emergency Management Agency (FEMA). The authors are solely responsible for the information contained herein. No liability for the information contained herein is assumed by Consortium of Universities for Research in Earthquake Engineering, California Institute of Technology, California Office of Emergency Services, or the Federal Emergency Management Agency.

The authors would like to thank Professor André Filiatrault, Manager of Element 1: Testing and Analysis, of the CUREE-Caltech Woodframe Project, for his support of this particular task within the Woodframe Project. In addition, Kelly Cobeen (Structural Engineer, GFDS Engineers) raised many insightful questions regarding practical issues related to the implementation of fluid dampers within woodframed buildings.
# Table of Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Preface</td>
<td>iii</td>
</tr>
<tr>
<td>Acknowledgments</td>
<td>vii</td>
</tr>
<tr>
<td>Table of Contents</td>
<td>ix</td>
</tr>
<tr>
<td>List of Tables</td>
<td>xi</td>
</tr>
<tr>
<td>List of Figures</td>
<td>xii</td>
</tr>
<tr>
<td>Summary</td>
<td>xvii</td>
</tr>
<tr>
<td>1. Introduction</td>
<td>1</td>
</tr>
<tr>
<td>1.1 Objectives</td>
<td>1</td>
</tr>
<tr>
<td>1.2 Scope</td>
<td>1</td>
</tr>
<tr>
<td>2. Literature Review on Innovative Seismic Protection Systems for Woodframed Buildings</td>
<td>3</td>
</tr>
<tr>
<td>2.1 Innovative Seismic Protection Systems</td>
<td>3</td>
</tr>
<tr>
<td>2.2 Base Isolation Systems for Light-Framed Wood Buildings</td>
<td>5</td>
</tr>
<tr>
<td>2.3 Supplemental Damping Systems for Light-Framed Wood Buildings</td>
<td>13</td>
</tr>
<tr>
<td>3. Model of Woodframed Shearwall</td>
<td>23</td>
</tr>
<tr>
<td>3.1 Description of Wall</td>
<td>23</td>
</tr>
<tr>
<td>3.2 Finite Element Model</td>
<td>24</td>
</tr>
<tr>
<td>3.3 Calibration of Shearwall Model</td>
<td>27</td>
</tr>
<tr>
<td>4. Supplemental Fluid Damper</td>
<td>31</td>
</tr>
<tr>
<td>4.1 Description of Damper</td>
<td>31</td>
</tr>
<tr>
<td>4.2 Dynamic Behavior of Damper and Mathematical Modeling</td>
<td>31</td>
</tr>
<tr>
<td>4.3 Damper Configuration within Shearwall</td>
<td>32</td>
</tr>
<tr>
<td>5. Effect of Fluid Damper on Woodframed Shearwall Behavior</td>
<td>35</td>
</tr>
<tr>
<td>5.1 Dynamic Properties of Shearwall</td>
<td>35</td>
</tr>
<tr>
<td>5.2 Seismic Response</td>
<td>36</td>
</tr>
<tr>
<td>5.2.1 Far-Field Earthquake Motion (Taft Record)</td>
<td>36</td>
</tr>
<tr>
<td>5.2.2 Near-Field Earthquake Motion (Newhall Record)</td>
<td>43</td>
</tr>
<tr>
<td>6. Model of Multi-Story Three-Dimensional Woodframed Building</td>
<td>49</td>
</tr>
<tr>
<td>6.1 Description of Building</td>
<td>49</td>
</tr>
<tr>
<td>6.2 Finite Element Model</td>
<td>51</td>
</tr>
</tbody>
</table>
7. Effect of Fluid Damper on Woodframed Building Behavior
   7.1 Dynamic Properties of Building
   7.2 Seismic Response
     7.2.1 Symmetric Building – Taft Earthquake Record
     7.2.2 Symmetric Building – Newhall Earthquake Record
     7.2.3 Asymmetric Building – Taft Earthquake Record
     7.2.4 Asymmetric Building – Newhall Earthquake Record

8. Implementation Issues and Recommendations for Further Study

9. Summary and Conclusions

10. References
# List of Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Table 1</strong></td>
<td>Summary of Studies Conducted on Advanced Seismic Protection Systems for Woodframed Structures.</td>
<td>4</td>
</tr>
<tr>
<td><strong>Table 2</strong></td>
<td>Values of Parameters Defining Sheathing Connection Model.</td>
<td>25</td>
</tr>
<tr>
<td><strong>Table 3</strong></td>
<td>Performance Levels for Light-Framed Wood Buildings.</td>
<td>40</td>
</tr>
</tbody>
</table>
List of Figures

Figure 1  Plan and Elevation Views of Two-Story Light-Framed Wood Building With an Elastomeric Base Isolation System (Adapted from Sakamoto et al. 1990).

Figure 2  Configuration of Elastomeric Bearings Used in Experimental Wood Building (Adapted from Sakamoto et al. 1990).

Figure 3  Hysteretic Behavior of Isolation System for Building With Multi-Stage Elastomeric Isolation Bearings (Adapted from Sakamoto et al. 1990).

Figure 4  Acceleration Response of Wood Building Subjected to Earthquake Ground Motion (Adapted from Sakamoto et al. 1990).

Figure 5  Photograph of Wood-Framed House With Sliding Base Isolation System (Adapted from Pall and Pall 1991).

Figure 6  Plan View of Wood-Framed House Showing Location of Sliding Isolation Bearings (Adapted from Pall and Pall 1991).

Figure 7  Photograph of Four-Story Wood-Framed Apartment Building Seismically Retrofitted With a Sliding Isolation System (Adapted from Zayas and Low 1997).

Figure 8  Close-Up View of Column Showing Isolation Bearing Installed Below Base Plate (Adapted from Zayas and Low 1997).

Figure 9  Cross-Section of Friction Pendulum System Bearing With Major Components Identified.

Figure 10  Relative Displacement Response Profiles for Fixed-Base and Base-Isolated Configurations (Adapted from Zayas and Low 1997).

Figure 11  Hysteretic Behavior of Conventional Light-Framed Wood Shearwall (Adapted from Filiatrault 1990).

Figure 12  Shearwall With Friction Dampers Located at Four Corners of Framing (Adapted from Filiatrault 1990).

Figure 13  Hysteretic Response of Friction Damper for Steady-State Sinusoidal Motion (Adapted from Filiatrault 1990).

Figure 14  Effect of Friction Dampers on the Hysteretic Behavior of a Wood-Framed Shearwall (Adapted from Filiatrault 1990).
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>Effect of Friction Dampers on the Energy Distribution Within a Wood-Framed Shearwall (Adapted from Filiatrault 1990).</td>
<td>16</td>
</tr>
<tr>
<td>16</td>
<td>Configuration of Conventional Light-Framed Wood Shearwall (Adapted from Dinehart et al. 1999).</td>
<td>17</td>
</tr>
<tr>
<td>17</td>
<td>Test Configurations for Shearwall With Viscoelastic Dampers (Adapted from Dinehart et al. 1999).</td>
<td>17</td>
</tr>
<tr>
<td>18</td>
<td>Detailed View of Four Viscoelastic Damper Test Configurations (Adapted from Dinehart et al. 1999).</td>
<td>18</td>
</tr>
<tr>
<td>19</td>
<td>Hysteresis Loops for Conventional Shearwall and Viscoelastically-Damped Shearwall (Adapted from Dinehart et al. 1999).</td>
<td>18</td>
</tr>
<tr>
<td>20</td>
<td>Effect of Viscoelastic Dampers on Energy Dissipation of Shear Walls (Adapted from Dinehart et al. 1999).</td>
<td>19</td>
</tr>
<tr>
<td>21</td>
<td>Fixed and Sliding Anchorage Connections for Hysteretic Damper (Adapted from Higgins 2001).</td>
<td>20</td>
</tr>
<tr>
<td>22</td>
<td>Experimental Testing Arrangement for Wood-Framed Shearwall with Hysteretic Dampers Along Diagonals (Adapted from Higgins 2001).</td>
<td>20</td>
</tr>
<tr>
<td>23</td>
<td>Reversed Cyclic Loading Response of Wall With and Without Hysteretic Dampers (Adapted from Higgins 2001).</td>
<td>21</td>
</tr>
<tr>
<td>24</td>
<td>Schematic of Wood Shearwall Used for Numerical Analysis (Nail Location Shown is Not Representative of Actual Nail Position).</td>
<td>23</td>
</tr>
<tr>
<td>25</td>
<td>Finite Element Model of Woodframed Shearwall.</td>
<td>24</td>
</tr>
<tr>
<td>26</td>
<td>Hysteretic Behavior of Sheathing Connection: (a) Static Cyclic Loading Protocol; (b) Response from Numerical Analysis and; (c) Experimental Test Data (Adapted from Dolan 1989)</td>
<td>26</td>
</tr>
<tr>
<td>27</td>
<td>Illustration of Sheathing Connection Parameters Defining the Monotonic and Cyclic Load-Displacement Behavior.</td>
<td>27</td>
</tr>
<tr>
<td>28</td>
<td>Calibration of Shearwall Finite Element Model via Static Monotonic Pushover Analysis with P-Delta Effects Removed (Experimental Test Data Taken from Dolan 1989).</td>
<td>28</td>
</tr>
</tbody>
</table>
Figure 29  Calibration of Shearwall Finite Element Model via Static Cyclic Analysis: (a) Static Cyclic Loading Protocol; (b) Hysteresis Loop from Numerical Analysis and; (c) Experimental Hysteresis Loop (Adapted from Dolan 1989).

Figure 30  Cross-Section of Fluid Damper with Major Components Identified.

Figure 31  Hysteretic Behavior of 8.9-kN (2-kip) Capacity Fluid Damper Subjected to Harmonic Motion at Three Different Frequencies.

Figure 32  Schematic of Fluid Damper and Orientation Within Shearwall.

Figure 33  Schematic of Alternate Fluid Damper Installation Configuration (Drawing Courtesy of Andre Filiatrault, University of California at San Diego, La Jolla, CA).

Figure 34  Fundamental and Second Mode Shape of Shearwall Without Dampers.

Figure 35  Ground Acceleration Corresponding to: (a) Taft Record – Lincoln School Tunnel (S69E Comp.) of 1952 Kern County Earthquake and (b) Newhall Record (90° Comp.) of 1994 Northridge Earthquake.

Figure 36  Acceleration Response Spectrum for Taft and Newhall Earthquake Records.

Figure 37  Hysteresis Loops of Shearwall Without and With Fluid Dampers Subjected to Taft Record (*plotted to same scale*).

Figure 38  Decomposition of Hysteresis Loop into Wall and Damper Contributions.

Figure 39  Drift Ratio for Wall With and Without Dampers When Subjected to Taft Record: (a) Time History for No Damper; (b) Time History for C = 87.6 kN-s/m and; (c) Summary of Peak Values.

Figure 40  Base Shear Coefficient for Wall With and Without Dampers When Subjected to Taft Record: (a) Time History for No Damper; (b) Time History for C = 87.6 kN-s/m and; (c) Summary of Peak Values.

Figure 41  Energy Distribution Within Wall for Taft Record: (a) No Damper and (b) With Damper.

Figure 42  Hysteresis Loops of Shearwall Without and With Fluid Dampers Subjected to Newhall Record (*not plotted to same scale*).

Figure 43  Hysteresis Loops of Shearwall Without and With Fluid Dampers Subjected to Newhall Record (*plotted to same scale*).
| Figure 44  | Decomposition of Hysteresis Loop into Wall and Damper Contributions. | 45 |
| Figure 45  | Time-Histories of Drift and Base Shear Response for Wall With and Without Damper Subjected to Newhall Record. | 46 |
| Figure 46  | Energy Distribution Within Wall for Newhall Record: (a) No Damper and (b) With Damper. | 47 |
| Figure 47  | Geometry of Building Model: (a) Isometric View of Both Symmetric and Asymmetric Building and (b) Plan Views of Symmetric and Asymmetric Building (Solid Rectangles Indicate Location of Shearwalls). | 50 |
| Figure 48  | Finite Element Model of Woodframed Building (Shear Panels Not Shown). | 52 |
| Figure 49  | Finite Element Models of: (a) Detailed Shearwall and (b) Simplified Shearwall. | 52 |
| Figure 50  | Comparison of Static Pushover Response for Detailed Wall Model and Simplified Wall Model. | 53 |
| Figure 51  | Comparison of Response for Detailed Wall Model and Simplified Wall Model When Subjected to Taft Record. | 53 |
| Figure 52  | Comparison of Time-Histories of Response for Detailed Wall Model and Simplified Wall Model When Subjected to Taft Record: a) Drift Ratio and b) Base Shear Coefficient. | 54 |
| Figure 53  | First Three Mode Shapes for Symmetric Building. | 55 |
| Figure 54  | Hysteresis Loops for Symmetric Building Without Dampers Subjected to Taft Record. | 56 |
| Figure 55  | Hysteresis Loops for Symmetric Building Without Dampers Subjected to Newhall Record. | 57 |
| Figure 56  | Hysteresis Loops for Symmetric Building With Dampers Subjected to Newhall Record. | 58 |
| Figure 57  | Energy Distribution Within Symmetric Building Subjected to Newhall Record. | 58 |
| Figure 58  | Hysteresis Loops for Asymmetric Building With and Without Dampers Subjected to the Taft Record. | 60 |
Figure 59  Hysteresis Loops for First Story of Asymmetric Building Without Dampers Subjected to Newhall Record.

Figure 60  Hysteresis Loops for First Story of Asymmetric Building With Dampers Subjected to Newhall Record.

Figure 61  Torsional Response of Second Story Floor Level of Asymmetric Building With and Without Dampers Subjected to Newhall Record.

Figure 62  Energy Distribution Within Asymmetric Building Subjected to Newhall Record.
Summary

For the 1994 Northridge, California Earthquake, it has been estimated that there was in excess of $20 billion dollars worth of damage to woodframed buildings. Such structures have generally received minimal attention from the earthquake engineering research community. One reason for this lack of attention is that such structures are considered to carry relatively small seismic forces due to their high strength-to-weight ratio and due to their inherent ability to dissipate energy. However, such structures have experienced significant damage during strong earthquakes to the extent that they are no longer capable of serving their function. Thus, there is a need for a cost-effective method by which light-framed wood buildings could be protected from strong earthquakes such that they remain intact with minimal damage.

The primary objective of this research project is to investigate the suitability of fluid viscous dampers for seismic protection of light-framed wood buildings. The objective was met by developing nonlinear finite element models of wood building components (shearwalls) and systems (three-dimensional houses) and performing numerical analysis to evaluate their seismic response with and without fluid dampers. In addition, some of the practical issues associated with implementation of fluid dampers within light-framed wood buildings have been explored. The results of the numerical study clearly demonstrate the ability of fluid dampers to dissipate a significant amount of seismic input energy, relieving the inelastic strain energy demand on the structure.
1. Introduction

The implicit performance level ascribed to structures designed according to seismic building code procedures corresponds to minimum life-safety criteria. Current thinking, however, is shifting away from this narrow point-of-view toward a seismic design philosophy in which multiple performance levels are considered. To achieve high performance levels for strong earthquakes generally requires the use of an innovative seismic protection system. For example, base isolation systems and supplemental damping systems are two innovative seismic protection systems that have seen a steadily increasing number of applications in large steel and concrete buildings over the past decade. However, to the authors’ knowledge, within the United States there is only one application of an innovative seismic protection system for woodframed structures.

The research discussed herein seeks to expand the number of applications of innovative seismic protection systems to woodframed structures by investigating the suitability of a supplemental energy dissipation system for seismic protection of woodframed structures. Specifically, the suitability of fluid dampers, which dissipate energy via orificing of a fluid, has been explored. A unique feature of the fluid dampers that have been studied is that they are capable of providing a very high-energy dissipation density (i.e., the energy dissipated is very large in comparison to the physical size of the damper). Thus, it is likely that the dampers could be conveniently located within the walls of a woodframed structure. This research project has been conducted in collaboration with an industry partner who has extensive experience in manufacturing fluid dampers for application in structural systems that require high-energy dissipation density. Since the industry collaboration increases the potential for near-term application of the research results, practical issues associated with the required capacity of such dampers and their installation within woodframed shearwalls have also been explored. To the knowledge of the authors, the research presented herein represents the first study on the application of fluid dampers within woodframed structures for seismic energy dissipation.

1.1 Objectives

The primary objective of this research project is to investigate the suitability of fluid dampers for seismic protection of light-framed wood buildings.

1.2 Scope

The objective was met by developing nonlinear finite element models of wood building components (shearwalls) and systems (three-dimensional houses) and performing numerical analysis to evaluate their seismic response with and without fluid dampers. In addition, some of the practical issues associated with implementation of fluid dampers within light-framed wood buildings have been explored.
2. Literature Review on Innovative Seismic Protection Systems for Woodframed Buildings

This section provides a literature review on the application of base isolation and supplemental damping systems for seismic protection of wood structures. The focus of the review is on the application of such advanced seismic protection systems to light-framed wood construction. The review reveals that both elastomeric bearings and sliding bearings have been considered for base isolation of light-framed wood buildings. Furthermore, both friction dampers, viscoelastic dampers, and hysteretic dampers have been studied for application to light-framed wood buildings. The reviewed literature clearly demonstrates that advanced seismic protection systems offer promise for enabling light-framed wood structures to resist major earthquakes with minimal damage. Note that the review contained herein represents a modified/updated version of a review that the principal author has published in CUREE Report No. W-03 (2001): Woodframe Project Testing and Analysis Literature Reviews.

2.1 Innovative Seismic Protection Systems

One approach to mitigating the effects of strong earthquakes on light-framed wood buildings is to incorporate an innovative seismic protection system within the building. For example, the response of the building can be de-coupled from the earthquake ground motion by introducing a flexible interface (i.e., a base isolation system) between the foundation and superstructure. The isolation system either shifts the fundamental period of the structure to a large value or limits the amount of force that can transferred to the structure such that interstory drifts (related to both structural and nonstructural damage) and floor and roof accelerations (related to contents damage) are reduced significantly. Alternatively, the seismic response of a building can be reduced by introducing a supplemental damping system within the framing of the building. The argument has been made that wood-framed buildings would not benefit from a supplemental damping system since the effective damping ratio of such structures is quite high (on the order of 7 to 15%). However, one must recognize that such levels of effective damping are the result of appreciable inelastic behavior associated with structural damage. A supplemental damping system would dissipate a portion of the seismic input energy, thereby reducing the amount of energy dissipated via inelastic behavior within the structural framing. The supplemental damping system dissipates a portion of the seismic input energy, reducing the amount of energy dissipated via inelastic behavior within the structural framing.

The number of applications of innovative seismic protection systems within buildings has been steadily growing within approximately the past fifteen years. Nearly all of these applications have been within either steel or concrete structures (e.g., see Soong and Constantinou, 1995). The two most common types of base isolation systems utilize either rubber bearings or sliding bearings between the foundation and superstructure. In contrast, there are a wide variety of supplemental damping systems available for implementation in buildings (Constantinou and Symans, 1993a and Constantinou et al., 1998). However, the most rapid growth in the application of supplemental damping systems to buildings has occurred for fluid dampers. Since the first experimental studies on a scale-model steel building frame in 1993 (Symans and Constantinou, 1993b), the number of implementations (completed and scheduled) of fluid dampers for suppression of dynamic response of bridge and building structures has grown to...
over 70. Although there are many factors that have contributed to this rapid growth, one of the primary reasons is the high-energy dissipation density of fluid dampers (i.e., fluid dampers are capable of dissipating a large amount of energy relative to their physical size).

Relatively few studies have been conducted on the application of innovative systems for seismic protection of wood frame structures. This may be partially due to a number of impediments to the implementation of base isolation and supplemental damping systems in woodframed structures. For example, the application of a base isolation system to a woodframed structure requires the introduction of a stiff diaphragm at the first floor, the introduction of flexible utility connections (e.g., gas, electric, telecommunications, etc.), educating the builder and/or building owner on the operation and expected performance of the protection system, and convincing the builder and/or the building owner of the value of the protection system relative to its cost. In addition, applications to new construction would require changing the typical construction schedule and standard practices of construction. For supplemental damping systems, some of the same issues apply and, in addition, a modified framing arrangement would be required. In spite of the aforementioned impediments, it is evident from the literature reviewed in this paper, a summary of which appears in Table 1, that advanced seismic protection systems offer promise for improving the seismic response of light-framed wood construction.

Table 1  Summary of Studies Conducted on Advanced Seismic Protection Systems for Woodframed Structures.

<table>
<thead>
<tr>
<th>Authors</th>
<th>Year</th>
<th>Topic</th>
<th>Type of Study</th>
</tr>
</thead>
<tbody>
<tr>
<td>Delfosse</td>
<td>1982</td>
<td>Elastomeric bearings applied to one-story residential building</td>
<td>Analysis</td>
</tr>
<tr>
<td>Reed and Kircher</td>
<td>1986</td>
<td>Elastomeric bearings and sliding bearings applied to five-story building</td>
<td>Analysis</td>
</tr>
<tr>
<td>Sakamoto et al.</td>
<td>1990</td>
<td>Elastomeric bearings applied to two-story residential building</td>
<td>- Analysis</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Experimental</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Implementation</td>
</tr>
<tr>
<td>Pall and Pall</td>
<td>1991</td>
<td>Flat sliding bearings applied to two-story residential building</td>
<td>- Analysis</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>- Implementation</td>
</tr>
<tr>
<td>Zayas and Low</td>
<td>1997</td>
<td>Friction pendulum system bearings applied to four-story steel/wood apartment building</td>
<td>- Analysis</td>
</tr>
</tbody>
</table>
2.2 Base Isolation Systems for Light-Framed Wood Buildings

Delfosse (1982)
Delfosse (1982) discusses issues associated with the application of base isolation systems to light (i.e., low weight) structures. It is noted that the design of an elastomeric bearing isolation system for light structures may be difficult due to the low mass of such structures and the requirement for a stiff wood floor or a concrete slab spanning between the isolators. As a rough approximation, assume that a light structure supported on an isolation system behaves as a rigid body having mass $m$. In this case, the natural period of the base-isolated structure is given by

$$T = 2\pi \sqrt{\frac{m}{k}}$$  \hspace{1cm} (1)

where $k$ is the lateral stiffness of the isolation system (assumed to behave elastically). Assuming the desired range of natural periods of base-isolated structures is from 1.5 to 4 seconds, the corresponding range of stiffness to mass ratios is:

$$2.5 < \frac{k}{m} < 17.5$$  \hspace{1cm} (2)

Thus, for structures with low mass (such as light-framed wood construction), the stiffness must be low enough to provide stiffness to mass ratios of less than about 20. Note that this relation between mass and stiffness is applicable to isolated structures in which the isolation system can be characterized by an elastic component which provides a restoring force. For isolation systems employing elastomeric bearings, the requirement for low stiffness tends to result in slender elastomeric bearings which may be prone to excessive shear strain and buckling. In spite of these apparent difficulties, Delfosse (1982) demonstrates, through an example design, that it is feasible to utilize an elastomeric base isolation system for a single-story woodframed house. The example design is a one-story wood-framed house that is rectangular in plan (12 m x 10 m) with a seismic mass of 29,770 kg. The elastomeric base isolation system is designed to protect the structure when it is subjected to a 5%-damped design spectrum corresponding to an earthquake having a Richter magnitude of 8 and a peak ground acceleration of 0.5 g. The design is intended to limit the spectral acceleration to 0.33 g so as to produce essentially elastic behavior of the structure and thus ensuring the life-safety of the occupants and protecting the building contents. The design results in a natural period of 1.4 sec for the base-isolated building. Note that no information is provided on the natural period of the fixed-base building (i.e., the building with a conventional foundation).

Reed and Kircher (1986)
Reed and Kircher (1986) discuss the seismic upgrade of a five-story woodframe building using two different isolation system configurations; one with elastomeric bearings and the other with flat sliding bearings. The building, constructed in 1886, is part of the Naval Postgraduate School in Monterey, California and has both historical and architectural significance to the U.S. Navy. For both isolation system configurations, a horizontal steel truss system was designed to stiffen the first floor so as to achieve rigid diaphragm action immediately above the isolation level. In
addition, a vertical truss system was designed to transfer loads between the isolation system and the woodframed superstructure. Furthermore, two restraint systems were developed; one to maintain rigidity under wind loading and the other to prevent building collapse under extreme earthquake loading. The isolation systems were designed to limit the forces transferred into the structure to 7% of the structure weight such that the structure remains essentially elastic under the design earthquake corresponding to a peak ground acceleration of 0.27 g. Thus, the wood framing above the first-floor required no modification, which is often an important consideration for the retrofit of structures having historical value. The fundamental natural period of the base-isolated structure that utilizes elastomeric bearings is 2.3 seconds. The natural period of the fixed-base building was not provided. The structure with the sliding isolation system configuration is characterized by a sliding coefficient of friction of about 1%. Numerical analyses of the existing and retrofitted (i.e., base-isolated) structures subjected to three earthquake time-histories, scaled to match the peak ground acceleration of the design spectrum, produced peak base shear response reductions ranging from 74% to 98%. Thus, the isolation systems appear to be very effective in terms of limiting the force transferred into the woodframed superstructure.

Sakamoto et al. (1990)
Sakamoto et al. (1990) present an experimental and analytical study of a two-story light-framed wood building supported on a base isolation system. The building was constructed at the University of Tokyo for in-field (not laboratory) experimental testing purposes. Plan and elevation views of the structure are shown in Figure 1. The footprint of the building is 7.28 m x 10.92 m and the total weight is 549 kN. The base isolation system consists of six laminated elastomeric bearings located along the perimeter of the foundation (see Figure 1). Note that, although not explained by Sakamoto et al. (1990), it is apparent from Figure 1 that the isolation bearings would need to be tied together via a stiff first floor diaphragm. Three different types of bearings were used in the experimental testing; namely, high damping rubber bearings, multi-stage rubber bearings, and lead-rubber bearings (see Figure 2). The hysteretic behavior of the isolation system for the building with multi-stage rubber bearings (Type B: Center bearing of Figure 2) is shown in Figure 3. All of the bearing types were designed to provide a fundamental natural frequency of 0.5 Hz (natural period of 2.0 sec) at a displacement amplitude of 15 cm, an equivalent viscous damping ratio of 15%, and an allowable deformation of 25 cm. The natural period of the fixed-base structure was not provided. For bearing displacements larger than 15 cm, a fail-safe support mechanism is activated in which the bottom of the first floor contacts the top of a rigid pedestal. Additional motion of the isolation system is impeded by sliding friction between the first floor and pedestal.
Figure 1 Plan and Elevation Views of Two-Story Light-Framed Wood Building With an Elastomeric Base Isolation System (Adapted from Sakamoto et al. 1990).

Figure 2 Configuration of Elastomeric Bearings Used in Experimental Wood Building (Adapted from Sakamoto et al. 1990).

Figure 3 Hysteretic Behavior of Isolation System for Building With Multi-Stage Elastomeric Isolation Bearings (Adapted from Sakamoto et al. 1990).

Free vibration tests of the isolated building in the direction of the short plan dimension revealed two dominant frequencies at about 0.8 Hz (first mode) and 8.5 Hz (second mode). The motion of the building with the multi-stage rubber bearings was also recorded during three different earthquakes. The strongest ground motion recorded and associated acceleration response of the
building in the direction of the short plan dimension (i.e., the X-direction of Figure 1) is shown in Figure 4. Note that the peak ground acceleration at the building site was 0.108g. The effectiveness of the isolation system is demonstrated by the reduction of acceleration (approximately 70%) transmitted from the ground level to the first floor. The dominant frequencies in the recorded acceleration response occurred at about 2.1 Hz and 9.7 Hz. The difference in fundamental frequencies for the free vibration response (0.8 Hz) and earthquake loading response (2.1 Hz) is attributed to the nonlinear behavior of the rubber bearings. The maximum displacement in the free vibration and earthquake loading tests was 15 cm and 0.1 cm, respectively. Since the effective stiffness of the bearings reduces with increasing displacement (see Figure 3), it is expected that the free vibration response (larger amplitude, lower stiffness) would be dominated by lower frequencies of motion. Finally, numerical simulations were performed for the building in a fixed-base and base-isolated configuration and subjected to the same earthquake as that described above. The numerical simulations were performed using a linear model of the building and isolation system in which effective system properties were determined by matching simulation results with the recorded seismic response of the building. The simulations indicated that the peak acceleration at the roof of the fixed-base structure would be about ten times as large as for the base-isolated structure.

![Figure 4 Acceleration Response of Wood Building Subjected to Earthquake Ground Motion (Adapted from Sakamoto et al. 1990).](image)

**Pall and Pall (1991)**

In 1988, a base isolation system was implemented within a two-story light-framed wood house in Montreal, Canada (Pall and Pall 1991). A photograph of the house is shown in Figure 5 and a plan view showing the location of the isolation bearings is shown in Figure 6. The house has two stories above grade and a basement below grade. The basement walls are reinforced concrete and the superstructure consists of light-framed wood construction with brick veneer. The isolation bearings used in this application are flat sliding bearings. A total of 15 bearings...
were installed along the perimeter of the basement wall. Note that the bearings were flat in the center with ramped surfaces along the edges (see Section 1-1 of Figure 6). The ramped surfaces provide increasing resistance at large displacements. The isolation gap is protected from the elements via flashing on the outside of the gap (see Section 1-1 of Figure 6). The coefficient of friction associated with the sliding interface of the bearings is 0.2. The coefficient of friction determines the maximum lateral force that can be transferred to the superstructure. In addition, the sliding friction dissipates energy as the bearing is cycled. Note that, strictly speaking, the concept of natural period does not apply to structures supported on flat sliding bearings and thus resonant response is not a concern.

![Figure 5](image.jpg)

*Figure 5* Photograph of Wood-Framed House With Sliding Base Isolation System  
(Adapted from Pall and Pall 1991).

Numerical analyses of the house in a fixed-base and base-isolated configuration were performed with earthquake ground motion represented by an artificial earthquake record. The earthquake record was developed to be consistent with the design response spectrum of the National Building Code of Canada. The results of the analyses show that the acceleration at the top of the structure is reduced by about 42% for the design peak ground acceleration of 0.18 g. This reduction in acceleration response is accompanied by a peak bearing displacement of 3 mm and a permanent displacement of 1.5 mm. Furthermore, as the peak ground acceleration increases, the effectiveness of the isolation system increases (i.e., the reductions in response acceleration are larger). Interestingly, the improved seismic performance did not reduce the cost of materials for construction of the house since standard size materials were used. Furthermore, the cost of the 15 isolators was $8,000. However, Pall and Pall (1991) suggest that the relatively low cost of friction base isolation bearings will result in widespread application of such bearings in low-rise construction including woodframed houses.
Zayas and Low (1997)
The implementation of sliding bearings in a four-story woodframed apartment building in San Francisco, California is discussed by Zayas and Low (1997). The building was severely damaged during the 1989 Loma Prieta earthquake and was retrofitted using sliding friction pendulum system (FPS) bearings. The four-story structure has a garage at the first story with apartments in the top three stories (see Figure 7). The damage to the first-story during the Loma Prieta earthquake was so severe that the entire first-story wood framing was replaced with a steel moment-resisting frame. The design of the frame was such that the structure exhibited essentially elastic behavior for a design earthquake with a peak ground acceleration of 0.21 g. The sliding bearings were installed under the base plates of each column of the steel frame (see Figure 8). Thus, the gravity loads of the structure are no longer transferred to the foundation over continuous sill plates but rather are transferred at the discrete locations of the bearings. The FPS bearings have a curved sliding surface with a constant radius of curvature (i.e., the sliding surface is circular in two dimensions and spherical in three dimensions) (see Figure 9). The motion of the structure when supported on the bearings is similar to that of a simple pendulum. The curved surface results in both energy dissipation due to sliding friction and energy storage due to the rising of the supported weight along the curved surface. In contrast to the behavior of
structures supported on flat sliding bearings, structures supported on curved bearings exhibit a natural period that can be determined by analogy to the natural period of a simple pendulum:

\[ T = 2\pi \sqrt{\frac{R}{g}} \]  

where \( R \) is the radius of curvature of the bearings, \( g \) is the acceleration due to gravity, and it has been assumed that the superstructure behaves as a rigid body. Interestingly, the natural period is independent of the weight supported by the bearings. Thus, the low mass of light-framed wood structures does not pose a problem for an isolation system that utilizes FPS bearings (recall that the potential for buckling of slender elastomeric bearings is indirectly due to the low mass supported by the bearing). The design of the bearings was based on a nonlinear, three-dimensional dynamic analysis using design earthquakes corresponding to the site-specific spectra for a magnitude 7 earthquake on the San Andreas Fault located 7 miles from the building site. The final design of the bearings resulted in a natural period of 2 seconds, a coefficient of friction of 10%, a displacement capacity of \( \pm 19 \) cm, and a required seismic gap of 28 cm. Note that no information was provided on the natural period of the fixed-base structure.

Figure 7  Photograph of Four-Story Wood-Framed Apartment Building Seismically Retrofitted With a Sliding Isolation System (Adapted from Zayas and Low 1997).

The performance of the base-isolated structure was evaluated via nonlinear time-history analysis. The results for input being a design-level earthquake with peak ground acceleration of 0.75g is presented here. Note that the design earthquake was considered to be representative of a site-specific spectrum corresponding to a magnitude 7 earthquake on the San Andreas fault (approximately 11.3 km from the site). Relative displacement response profiles are compared in Figure 10 for two cases: 1) fixed-base configuration (but with first story strengthened by steel moment-resisting frame) and 2) isolated configuration. As expected, the displacement response of the isolated building is concentrated at the isolation level with very little interstory drift response in the superstructure. In contrast, the non-isolated building experiences significant interstory drift that is concentrated in the second story. The effect of the isolation system is to reduce the peak interstory drift by about 95%. The isolated building remains entirely elastic.
while the non-isolated building experiences a ductility demand of 38.4 for the second story. Zayas and Low (1997) suggest that such a high ductility demand is beyond the reasonable capacity of the wood-stucco frame and thus there is a significant risk of collapse of the non-isolated building. In summary, the numerical analyses demonstrate that the isolated structure would respond elastically if subjected to a magnitude 7 earthquake on the nearby San Andreas fault. Finally, it is noted that retrofitting the building using a seismic isolation system was cost effective in that its use avoided the need to add shearwalls over the height of the structure and to remove and replace ornate finish materials having architectural significance. The cost of adding the FPS bearings and the seismic gap details was $35.50 per m².

Figure 8 Close-Up View of Column Showing Isolation Bearing Installed Below Base Plate (Adapted from Zayas and Low 1997).

Figure 9 Cross-Section of Friction Pendulum System Bearing With Major Components Identified.
2.3 Supplemental Damping Systems for Light-Framed Wood Buildings

Filiatrault (1990)
The seismic response of friction-damped timber shearwalls has been studied by Filiatrault (1990). The cyclic lateral force-displacement relation for conventional light-framed shearwalls exhibits both a progressive loss of stiffness and pinching of the hysteresis loops (see Figure 11). Such behavior is a direct result of damage to the shearwall. Note that, due to the pinching of the hysteresis loops, the shearwall must experience large deformations in order to dissipate the seismic input energy. Filiatrault (1990) proposed the concept of installing supplemental energy dissipation elements at the four corners of a shearwall (see Figure 12). The elements are friction dampers which absorb a significant portion of the seismic input energy, reducing the amount of energy that needs to be dissipated by the framing via inelastic behavior. The friction dampers consist of a slotted slip joint that dissipates energy via friction as the two sides of the joint slide with respect to each other. Sliding motion of the slip joint is induced by deformation of the four corners of the shearwall. Thus, the wood framing of the shearwall must deform to activate the friction dampers. However, since the friction dampers absorb a portion of the seismic energy input, the energy dissipation demand on the framing will be reduced. Note that the triangular configuration of the friction damper (see Figure 12) is beneficial in maintaining the integrity of the woodframed corners of the shearwall. The stability of the mechanical properties of the friction damper are demonstrated in Figure 13 which shows the repeatability of the force-displacement relation for a single damper subjected to 50 cycles of sinusoidal motion at a frequency of 0.2 Hz and an amplitude of ± 1.52 cm.
Numerical analysis of a friction-damped shearwall (2.4 m x 2.4 m) was performed using nonlinear models for the shearwall and the friction dampers. The shearwall consisted of 38 x 89 mm spruce-pine-fir stud framing with center-to-center spacing of 600 mm and 9 mm thick plywood sheathing. The model of the shearwall accounts for both stiffness degradation and pinching of the hysteresis loops. Experimental test data obtained from shaking table tests of a shearwall without friction dampers (Dolan 1989) demonstrated the suitability of the analytical model of the shearwall. A comparison of the hysteretic behavior of the shearwall with and without the supplemental friction dampers and subjected to the El Centro record (component NS) of the 1940 Imperial Valley Earthquake is shown in Figure 14. The friction dampers improve
the seismic performance by reducing the peak force and displacement at the top of the wall and essentially eliminating the pinching of the hysteresis loops. The effectiveness of the friction dampers is also demonstrated in the energy time histories shown in Figure 15. There is a marked reduction in the hysteretic energy dissipation demand on the shearwall. Furthermore at the end of the earthquake, approximately 60% of the input energy is dissipated by the friction dampers.

Figure 13 Hysteretic Response of Friction Damper for Steady-State Sinusoidal Motion (Adapted from Filiatrault 1990).

Figure 14 Effect of Friction Dampers on the Hysteretic Behavior of a Wood-Framed Shearwall (Adapted from Filiatrault 1990).
The dynamic behavior of light-framed wood shearwalls with viscoelastic dampers was experimentally evaluated by Dinehart and Shenton (1998) and Dinehart et al. (1999). The experimental tests were conducted on 2.44 m x 2.44 m (8 ft x 8 ft) shearwalls framed with 38 mm x 89 mm (2 in. x 4 in., nominal) interior studs spaced at 40.6 cm (16 in.) on center. Two 11.9 mm (15/32 in.) thick plywood sheets were used as sheathing panels (see Figure 16). Four different damper configurations were investigated (see Figure 17 and 18). In each configuration, the viscoelastic damper dissipates energy via shearing action of a viscoelastic rubber-like material. The shearing action is induced by relative motion between the two ends of the damper at their points of attachment to the shearwall. Thus, the shearwall must deform for the viscoelastic dampers to dissipate energy. Note that, in addition to the corner damper configuration shown in Figure 17(a) where the dampers are located in the top corners, a separate configuration with the dampers located in the bottom corners was investigated.

The shearwalls were supported at the bottom plate and a prescribed lateral displacement time history was imposed at the top plate. The prescribed displacement time history consisted of 72 cycles of varying displacement amplitude (specifically, the sequential phased displacement test protocol was used). The hysteretic behavior for the conventional wall (no dampers) and the wall with two different viscoelastic damper configurations (sheathing-to-stud and diagonal bracing) is shown in Figure 19. Due to the viscoelastic nature of the dampers, both the effective stiffness and energy dissipation increased by adding the dampers to the wall. However, as a result of utilizing a prescribed displacement time history, the hysteresis loops are not dramatically different for the conventional and viscoelastically damped walls. Although the hysteresis loops are not very different, the cumulative energy dissipated by the conventional and viscoelastically damped walls is significantly different (e.g., the cumulative energy dissipated up to the displacement corresponding to the ultimate load was approximately 55% larger for the viscoelastically damped wall with the damper along the diagonal as compared to the conventional wall). It should also be understood that the conventional walls must dissipate all of the energy.
the input energy, primarily via inelastic behavior of the wall. In contrast, for the viscoelastically damped walls a large portion of the input energy is dissipated by the viscoelastic dampers and thus the energy dissipation demand on the wall itself is reduced significantly.

![Figure 16 Configuration of Conventional Light-Framed Wood Shearwall](Adapted from Dinehart et al. 1999).

![Figure 17 Test Configurations for Shearwall With Viscoelastic Dampers](Adapted from Dinehart et al. 1999).
A comparison of energy dissipation per cycle for three shearwall configurations is shown in Figure 20. The energy dissipation is shown for four cycles of motion at a displacement amplitude equal to the so-called first major event (i.e., ±1.80 cm). Note that the energy dissipation capacity of both the conventional and viscoelastically damped shearwalls decreases as the number of cycles increases. However, the degradation for both configurations is due to the inelastic behavior of the shearwall (since the reduction in energy dissipation from the one cycle to the next is approximately equal for both configurations). Thus, the viscoelastic dampers
provide a stable source of energy dissipation during cyclic motion of the shearwall. A stable source of energy dissipation is particularly important in terms of the ability of a woodframed structure to resist strong earthquake aftershocks.

Dinehart and Lewicki (2001) experimentally evaluated the performance of a full-scale (2.4 m x 2.4 m) wood shearwall with a thin (0.5 mil) viscoelastic sheet material located at the interface between the framing and sheathing. The sheet material is similar to double-sided tape in that it adheres to both the framing and sheathing. The wall was tested using the Sequential Phased Displacement test protocol at a frequency of 1 Hz and a maximum displacement of 76 mm. Similar to the tests performed by Dinehart and Shenton (1998) and Dinehart et al. (1999) wherein discrete viscoelastic dampers were utilized, the viscoelastic sheeting increased the effective stiffness and energy dissipation capacity of the wall. In addition, the viscoelastic sheeting provided a stable source of energy dissipation during cyclic motion of the shearwall.

Higgins (2001)
The application of a hysteretic damper to a woodframed shearwall is investigated by Higgins (2001). The hysteretic damper consists of a diagonal brace with a fixed anchorage at the top corner of the wall and a sliding anchorage at the opposite corner (see Figure 21). In compression, the diagonal rod is allowed to slip through the sliding anchorage, eliminating the possibility of buckling. In tension, the diagonal rod is gripped by the sliding anchorage. The damper behavior has the apparent effect of expanding the kinematic surface of the element and thus the damper is called a “kinematically expanding hysteretic damper.” Note that the term “hysteretic” is used to describe the damper since its behavior is displacement-dependent rather than velocity-dependent.
Experimental tests were performed in which a woodframed shearwall was outfitted with two of
the dampers; one along each diagonal (see Figure 22). The response of the wall with and without
the damper when subjected to reversed cyclic loading of increasing displacement amplitude is
shown in Figure 23. Figure 23 demonstrates that the dampers were effective in reducing the
strength and stiffness degradation of the wall and increasing the energy dissipation capacity. In
addition, the characteristic pinching behavior of woodframed shearwalls is virtually absent for
the wall with the damper. The dampers eventually failed by fracture on the gross cross-section
of the diagonal rod.

Figure 21  Fixed and Sliding Anchorage Connections for Hysteretic Damper (Adapted from Higgins 2001).

Figure 22  Experimental Testing Arrangement for Wood-Framed Shearwall with Hysteretic Dampers
Along Diagonals (Adapted from Higgins 2001).
A numerical study is also presented in which a simple structure is subjected to an earthquake record. The study indicates that the proposed damper is more effective in reducing peak displacements than both ordinary tension-only bracing and ADAS/UBB (Added Damping and Stiffness or Unbonded Brace) devices.
3. Model of Woodframed Shearwall

3.1 Description of Wall

The wall model developed herein is similar to a series of walls that were experimentally tested by Dolan (1989). The dimensions of the shearwall were 2.44 m x 2.44 m (8 ft x 8 ft) (see Figure 24). The framing of the wall consisted of 38.1 mm x 88.9 mm (nominal 2 in. x 4 in.) lumber with the vertical studs spaced at 60.96 cm (24 in.) on center. Single end studs and sill/sole plates were used. The wall was sheathed with 1.22 m x 2.44 m (4 ft x 8 ft) plywood sheathing panels having a thickness of 9.53 mm (3/8 in.). The connections between the sheathing and framing consisted of 6.35 cm (2.5 in.) 8d galvanized common nails with field and perimeter nail spacing of 15.24 cm (6 in.). The weight at the top of the wall was 44.5 kN (10 kips), which is intended to represent the tributary weight if the wall were located at the first story of a three-story apartment building in North America. The weight was distributed at the nodes along the top plate (i.e., at the top of each stud). The bottom sill plate was assumed to be fixed to the foundation. Thus, the effects of uplift are not considered. It should be noted that the wall depicted in Figure 24 does not include finish materials. As noted by Fischer et al. (2001) [Task 1.1.1 of the Woodframe Project] and Gatto and Uang (2002) [Task 1.3.1 of the Woodframe Project], the finish materials used on structural walls and partitions can make a considerable difference in system behavior. The inclusion of finish materials increases the stiffness of wood framing systems which generally reduces the drift ratios.

![Schematic of Wood Shearwall Used for Numerical Analysis (Nail Location Shown is Not Representative of Actual Nail Position).](image_url)
3.2 Finite Element Model

A nonlinear finite element model of a wood-framed shearwall was developed for the numerical analyses using the commercial program ABAQUS (ABAQUS, 1998) (see Figure 25). A total of 415 nodes were used to define the model. The framing members and sheathing panels were modeled as two-dimensional isoparametric, isotropic, elastic cubic beam elements and two-dimensional isoparametric, orthotropic, elastic linear plane stress elements, respectively. Elastic framing members and sheathing panels are used since, as is commonly understood, the hysteretic behavior of the wall is considered to be dominated by the hysteretic behavior of the sheathing connections. The initial clearance between the sheathing panels was assumed to be 7.94 mm (5/16 in.). The interaction between the two sheathing panels was accounted for via a softened contact pressure-clearance relationship with an exponential law.

![Finite Element Model of Woodframed Shearwall.](image)

The sheathing connections associated with each nail were modeled as two orthogonal (one vertical and one horizontal) uncoupled nonlinear springs whose characteristics were defined by a modified hybrid Stewart-Dolan connector model consisting of a series of straight-line segments (Stewart 1987) and an exponential backbone curve (Dolan 1989). The hysteresis loop for the connection model is shown in Figure 26(b) for the static cyclic loading protocol shown in Figure 26(a). For qualitative comparison, a hysteresis loop obtained from experimental testing of a similar sheathing connection is shown in Figure 26(c). The idealized hysteretic loop shown in Figure 26(b) is generated using a rule-based model for loading/unloading behavior. The connector model includes an exponential backbone curve which represents the monotonic resistance to lateral displacement and serves as an envelope for the force developed during cyclic motion. In addition, the model includes linear loading/unloading segments and a linear pinching region. The backbone curve is given by the following expression relating the connection shear force, \( P \), and the connection displacement, \( u \):
where the parameters defining the hysteretic behavior are given by (see Figure 27):

\[ P = (P_o + K_2 u) \left[ 1 - \exp \left( \frac{-K_3 u}{P_o} \right) \right] \]  \hspace{1cm} (3)

Four of the seven parameters of the sheathing connection model \((P_o, P_1, K_o, \text{ and } K_2)\) were obtained from experimental test data (see Dolan 1989). The other three parameters \((K_3, K_4, \text{ and } u_{ult})\) were determined via a calibration procedure in which experimental monotonic and cyclic test data was compared with the results of numerical analyses (see Section 3.3 below). The parameters defining the sheathing connection model are summarized in Table 2.

### Table 2 Values of Parameters Defining Sheathing Connection Model.

<table>
<thead>
<tr>
<th>(P_o)</th>
<th>(P_1)</th>
<th>(K_o)</th>
<th>(K_2)</th>
<th>(K_3)</th>
<th>(K_4)</th>
<th>(u_{ult})</th>
</tr>
</thead>
<tbody>
<tr>
<td>915 N</td>
<td>180 N</td>
<td>1320 N/mm</td>
<td>39 N/mm</td>
<td>-3.0 N/mm</td>
<td>29.5 N/mm</td>
<td>15.24 mm</td>
</tr>
<tr>
<td>(206 lb)</td>
<td>(40 lb)</td>
<td>(7536 lb/in)</td>
<td>(220 lb/in)</td>
<td>(-17.1 lb/in)</td>
<td>(168.4 lb/in)</td>
<td>(0.6 in.)</td>
</tr>
</tbody>
</table>

Note: Values of \(P_o, P_1, K_o, \text{ and } K_2\) taken from experimental data (Dolan, 1989) which represents average results from static-monotonic, static-cyclic, and dynamic cyclic tests of plywood sheathing connections with framing grain parallel and sheathing grain perpendicular to load.

The pinched shape of the hysteresis loops of Figure 26(b) for small displacements is due to the crushing of the wood sheathing and framing along with nail yielding as the connections are cycled. Note that the commonly observed strength degradation and stiffness degradation for repeated cycles at the same displacement was neglected. Beyond the displacement \(u_{ult}\) (corresponding to the ultimate load \(P_{ult}\)) the nail begins to withdraw from the framing and sheathing, resulting in a rapid reduction in load-carrying capacity. Thus, in this study, it is assumed that the connection has essentially failed if the displacement exceeds \(u_{ult}\). Any possible failure from nail tear-out has not been considered. It should be noted that the simplifications mentioned above were made for purposes of efficient analysis and were deemed acceptable since the purpose of this study was to demonstrate the effects of supplemental damping in a representative woodframed shearwall, not to accurately model all of the details associated with the dynamic behavior of the shearwall.
Figure 26 Hysteretic Behavior of Sheathing Connection: (a) Static Cyclic Loading Protocol; (b) Response from Numerical Analysis and; (c) Experimental Test Data (Adapted from Dolan 1989).
3.3 Calibration of Shearwall Model

Since the shearwall model was not developed with the intent of accurately reproducing the behavior of any particular shearwall, a calibration procedure was performed so as to ensure that the monotonic and cyclic behavior of the wall was reasonable. The shearwall model was calibrated by adjusting the value of the three sheathing connection parameters $K_3$, $K_4$, and $u_{ult}$ so as to obtain a reasonable match between static monotonic and static cyclic experimental test data with results from numerical analyses. Note that the stiffness of the softening region, $K_3$, was selected to be equal to one-tenth of the stiffness of the pinching region, $K_4$, to avoid numerical difficulties associated with abrupt changes in stiffness at the ultimate displacement.

The static monotonic analysis was performed in a manner similar to the ASTM E564-76 Standard in which a linearly increasing displacement is applied at the top sill plate while holding the bottom sill plate fixed (see Figure 28). Similarly, a static cyclic analysis was performed using the static cyclic loading protocol shown in Figure 29(a). Note that in the experimental static cyclic test, the duration of the loading protocol was 45 minutes whereas the duration shown in Figure 29(a) is 25 seconds. For experimental testing, the load rate affects the wall behavior. However, for the numerical analysis, the load rate did not affect the wall behavior since the analysis was specified to be performed as a static analysis in which the inertia and damping forces were not considered. As shown in Figure 28 and 29, the comparison between the analytical and experimental data is reasonably good for both monotonic and cyclic motion, indicating that the parameters of the model are adequate for capturing the salient behavior of a plywood-sheathed shearwall. Note that, for purposes of practical experimental testing, the tributary mass was only effective in the horizontal direction (i.e., inertia forces were applied to the wall by the tributary mass but the P-Delta effects from the inertial mass were extracted from the experimental data). Thus P-Delta effects were not considered in either the static monotonic or cyclic analyses.
Figure 28 Calibration of Shearwall Finite Element Model via Static Monotonic Pushover Analysis with P- Delta Effects Removed (Experimental Test Data Taken from Dolan 1989).
Figure 29 Calibration of Shearwall Finite Element Model via Static Cyclic Analysis: (a) Static Cyclic Loading Protocol; (b) Hysteresis Loop from Numerical Analysis and; (c) Experimental Hysteresis Loop (Adapted from Dolan 1989).
4. Supplemental Fluid Damper

4.1 Description of Damper

As mentioned previously, fluid viscous dampers were selected for investigation in this study primarily due to their high energy dissipation density (i.e., their ability to dissipate large amounts of energy in comparison to their size), a feature that is important due to the relatively narrow confines of a typical woodframed shearwall. A cross-sectional view of a typical fluid damper is shown in Figure 30. The damper consists of a cylinder filled with low-viscosity fluid (silicone oil). The fluid is forced to pass through orifices within the piston head as the attached piston rod is cycled. Due to the relatively small size of the piston head orifices, the fluid passes through the orifices at high speeds, resulting in the development of heat energy which is transferred to the environment via convection and conduction. The rod make-up accumulator shown in Figure 30 accounts for the volume of fluid displaced by the piston rod as it enters the cylinder, thus minimizing the development of stiffness due to fluid compression (Symans and Constantinou 1998). Thus, the dampers behave essentially as pure energy dissipation devices. The design of structures that incorporate such dampers becomes simplified since the dampers may be regarded as simply adding additional energy dissipation capacity to the structure. Of course, one must recognize that the installation of supplemental dampers will alter the load path for the transfer of forces within the structure.

![Figure 30 Cross-Section of Fluid Damper with Major Components Identified.](image)

4.2 Dynamic Behavior of Damper and Mathematical Modeling

The hysteretic behavior of an 8.9-kN (2-kip) capacity fluid damper subjected to sinusoidal input motion is presented in Figure 31 for room temperature conditions (23°C) and frequencies of 1, 2 and 4 Hz (Symans and Constantinou 1998). As is evident from Figure 31, in this range of frequencies, typical of the fundamental mode of light-framed residential wood construction, the fluid damper exhibits insignificant restoring force and its behavior is essentially linear viscous (i.e., the shape of the hysteresis loops is essentially elliptical with negligible slope). Thus, an appropriate mathematical model for the damper is given by that of a simple linear viscous dashpot:

\[ P(t) = C \ddot{u}(t) \] (4)
where \( P(t) \) is the damper force, \( C \) is the damping coefficient, and \( \dot{u} \) is the velocity of the piston head with respect to the cylinder. Note that, in spite of the relatively low-viscosity of the silicone oil within the damper, such dampers are often referred to as *fluid viscous dampers*, giving the impression that the fluid within the damper is highly viscous. The term fluid viscous damper is the result of the hysteretic behavior of the damper with is equivalent to a linear viscous dashpot. The hysteretic behavior is primarily controlled by the shape of the piston head orifices, not the viscosity of the fluid. In fact, the shape of the piston head orifices is often altered to produce nonlinear viscous dashpot behavior as given by:

\[
P(t) = C|\dot{u}(t)|^\alpha \text{sgn}[\dot{u}(t)]
\]  

(5)

where the velocity exponent \( \alpha \) varies from about 0.5 to 2 and \( \text{sgn}(\dot{u}) \) is the signum function. At a given frequency and amplitude of harmonic motion, the nonlinear damper with velocity exponent of 0.5 produces a more rectangular loop than a linear damper, resulting in an increase in energy dissipation capacity of approximately 11% (Symans and Constantinou 1998). In this study, we focus on the linear damper model. It is anticipated that future studies by the authors will explore the effects of nonlinear dampers.

**Figure 31** Hysteretic Behavior of 8.9-kN (2-kip) Capacity Fluid Damper Subjected to Harmonic Motion at Three Different Frequencies.

### 4.3 Damper Configuration within Shearwall

The authors believe that fluid dampers offer considerable promise for application within woodframed buildings due to their high-energy dissipation density, which allows the dampers to be conveniently located within the walls of a woodframed structure. For example, in this study, the damper was positioned along the diagonal of the wall (see Figure 32). In this configuration, dual let-in rods are used to connect the lower corner of the wall to one end of the damper. One rod is located on each side of the wall and small metal straps are used to prevent the rod from buckling outward. One advantage to this configuration is that the damper force lies concentrically within
the plane of the wall and thus there are no bending moments applied to the wall at the corner connections. Furthermore, this configuration maintains a certain level of gravity load carrying capacity and does not interfere with the installation of sheathing on either side of the wall. However, a disadvantage to this configuration is that the effectiveness of the damper is reduced by 50% due to the diagonal orientation. Although the damper shown in Figure 32 is located in the upper corner of the wall, this is not necessary; the damper may be just as easily positioned at the lower corner.

Figure 32  Schematic of Fluid Damper and Orientation Within Shearwall.

An alternate installation configuration, as originally conceived by Professor Daniel Dolan of Virginia Polytechnic Institute and State University and refined by Professor Andre Filiatrault of University of California at San Diego, involves a re-orientation of the studs. As shown in Figure 33, the studs are rotated 90° with two studs located at each original stud location. The damper is located near one of the lower corners of the wall and is connected, through a steel tube, to the opposite upper corner of the wall. Wood spacers are used to connect the two rows of side studs in order to increase their lateral stability under gravity loads. The tube fits snugly between the studs so as to inhibit lateral buckling of the tube. Special steel anchorages are utilized at the lower and upper corners of the wall to effectively transfer the damper force through the wall and into the foundation. Note that this bracing configuration does not interfere with the transfer of gravity loads or with the placement of sheathing on both sides of the framing.
Figure 33  Schematic of Alternate Fluid Damper Installation Configuration
(Drawing Courtesy of Andre Filiatrault, University of California at San Diego, La Jolla, CA).
5. Effect of Fluid Damper on Woodframed Shearwall Behavior

5.1 Dynamic Properties of Shearwall

The dynamic properties of the wall were determined via an undamped eigenvalue analysis using the initial elastic stiffness of the wall. The resulting natural frequencies in the fundamental and second modes of vibration were 4.18 and 22.8 Hz, respectively, with corresponding mode shapes as shown in Figure 34. Note that the mode shapes shown in Figure 34 include deformations of both the sheathing panels and framing. The inherent damping in the wall was accounted for via a Rayleigh damping formulation wherein, for low amplitudes of vibration, the damping ratio in the first two modes of vibration was assumed to be 2% and 8%, respectively.

For the wall with a fluid damper installed, the fundamental frequency is essentially unchanged since the damper may be regarded as providing a source of pure energy dissipation and thus does not increase the stiffness of the wall. Two different dampers were considered in this study, one having a damping coefficient of 17.5 kN-s/m (100 lb-s/in) and the other having a damping coefficient of 87.6 kN-s/m (500 lb-s/in). Considering a simplified SDOF representation of the wall, wherein the fundamental frequency is taken as 4.18 Hz (as given above) and the lumped weight is 44.5 kN, the addition of the damper results in an increase in the fundamental mode damping ratio from 2% to 5.7% and 20.4% for \( C = 17.5 \text{ kN-s/m} \) and 87.6 kN-s/m, respectively. Note that the smaller damping coefficient was selected since the principal author had previously performed experimental testing on such a damper and recognized that the size of the damper was such that it could readily be installed in a woodframed shearwall (see Figure 31 for cyclic test data obtained by the principal author on the aforementioned damper). The larger damping coefficient was selected after it was recognized that the smaller capacity damper was not adequate for resisting strong near-field motions. Furthermore, it was confirmed (see Section 8) that the damper with the larger damping coefficient could be readily manufactured at a suitable size for installation within a woodframed shearwall.

![Figure 34](image.jpg)

**Figure 34** Fundamental and Second Mode Shape of Shearwall Without Dampers.
5.2 Seismic Response

The mathematical model of the shearwall was subjected to the following two historical earthquake ground motions:

1) 1952 Kern County Earthquake, Taft record – Lincoln School Tunnel (S69E comp.)

2) 1994 Northridge Earthquake, Newhall record – LA County Fire Station (90° comp.)

For regions of California that are regarded as having a high seismic risk, the probability of exceedance for the Taft record would be on the order of 50% in 50 years which may be regarded as a relatively frequent event. For the Newhall record, the probability of exceedance would be on the order of 10% in 50 years which may be regarded as a code level event. The recorded acceleration for these two ground motions for the first 30 seconds is shown in Figure 35 and the 5%-damped acceleration response spectra (actual, not pseudo) are shown in Figure 36. The two records were selected since, as is readily evident in Figure 36, the records are disparate (i.e., the Taft record is a weak, far-field motion while the Newhall record is a strong, near-field motion). The fluid dampers proved to be beneficial for both types of ground motion in that, for the weak, far-field motion, the shearwall remained essentially elastic with little to no permanent damage while for the strong, near-field motion, the shearwall was damaged but much less so than without the dampers.

5.2.1 Far-Field Earthquake Motion (Taft Record)

The analyses for the Taft earthquake record were performed for the shearwall alone and for the shearwall with one fluid damper installed along the diagonal (see Figure 32). The damper was assumed to exhibit linear viscous behavior as given by Eq. (4) with a damping coefficient of either 17.5 kN·s/m (100 lb·s/in) or 87.6 kN·s/m (500 lb·s/in). Initially, we concentrate on the case of a damping coefficient of 87.6 kN·s/m. Recall from Section 5.1 that such a damper increases the equivalent viscous damping ratio in the fundamental mode from 2% to 20.4%. Note that, due to limited computer storage capacity, the response of the wall was determined for only the first 15 seconds of the Taft earthquake record shown in Figure 35(a). This limitation was considered to be acceptable since the strongest portion of the record occurs within the first 15 seconds.

The effectiveness of the damper is clearly depicted in the hysteresis loops shown in Figure 37 where the two loops are plotted to the same scale. For this relatively weak earthquake, the wall alone (i.e., no damper) experiences inelastic behavior. Although the inelastic response is not strong, the wall has been weakened in terms of resisting aftershocks and future earthquakes. In contrast, the wall with the damper experiences significantly smaller drifts (peak reduction of 54%) and shear forces (peak reduction of 34%), implying much less damage to the wall. The peak damper force, velocity and stroke are 3.87 kN (868.6 lb), 4.41 cm/s (1.74 in/s) and 0.265 cm (0.104 in.), respectively. As discussed in Section 8, a damper with such behavior can be readily manufactured. Furthermore, one may note the somewhat larger forces that develop within the small displacement region of the pinching zone for the case of the wall with the damper. This is the result of the damper force being proportional to velocity, thus leading to large damper forces in the region of the pinching zone where the displacements are small and the velocities are large. The development of larger forces in the pinching region is one of the
reasons that the dampers are so effective. Note that, for the wall with the damper, the hysteresis loop consists of a combination of damper behavior and wall behavior and thus it is not readily apparent how the wall itself (i.e., the wood framing system) performed. However, as shown in Figure 38, the hysteresis loop can be decomposed into its contributions from the wall and damper, allowing one to visualize the performance of the wall itself.

![Graph](image)

**Figure 35** Ground Acceleration Corresponding to: (a) Taft Record – Lincoln School Tunnel (S69E Comp.) of 1952 Kern County Earthquake and (b) Newhall Record (90° Comp.) of 1994 Northridge Earthquake.
Figure 36  Acceleration Response Spectrum for Taft and Newhall Earthquake Records.

Figure 37  Hysteresis Loops of Shearwall Without and With Fluid Dampers Subjected to Taft Record (plotted to same scale).
The potential for damage to the wall can be conveniently characterized by the drift ratio. According to the *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (FEMA, 1997), for light-framed wood construction, the transient drift ratios corresponding to the structural performance levels of Immediate Occupancy (IO), Life-Safety (LS), and Collapse Prevention (CP) are 1%, 2% and 3%, respectively (see Table 3). A description of the damage state for these performance levels is also provided in Table 3. As mentioned in Section 3.1, the shearwall analyzed herein does not include the effects of finish materials. The inclusion of finish materials increases the stiffness of wood framing systems which generally reduces the drift ratios.

The time-history of drift ratio for the wall without a damper and with a damper having a damping coefficient of $C = 87.6 \text{ kN-s/m}$ is shown plotted to the same scale in Figure 39(a) and (b), respectively. Evidently, even for the wall without dampers, the Immediate Occupancy objective is met. However, as noted above, there is more damage to the wall. The peak drift ratio for the two time-histories shown in Figure 39(a) and (b), as well as the case of the wall with a damper having a damping coefficient of $C = 17.5 \text{ kN-s/m}$, are plotted in Figure 39(c). Recall from Section 5.1 that the fundamental mode damping ratio for the wall without a damper is 2%. For the two cases with a damper, the damping ratio based on a SDOF idealization of the wall is 5.7% and 20.4% for $C = 17.5 \text{ kN-s/m}$ and $87.6 \text{ kN-s/m}$, respectively. Thus, an increase in the
The base shear coefficient (i.e., the base shear normalized with respect to the weight of the wall) is another measure of the wall performance. The time-histories of base shear coefficient (plotted to the same scale) and associated peak values are shown in Figure 40. An increase in the damping ratio from 2% to 5.7% (a 185% increase) results in a 14.8% reduction in peak base shear while an increase in the damping ratio from 5.7% to 20.4% (a 258% increase) results in an additional 19.2% reduction in peak base shear.

The performance of the fluid dampers may also be evaluated by considering the energy distribution within the wall during the earthquake. The time histories of various energy quantities are shown in Figure 41 for the wall with and without the fluid damper having a damping coefficient of \( C = 87.6 \text{ kN-s/m} \). Note that the two plots shown are plotted to the same scale. Figure 41(a) indicates that, without a fluid damper, essentially all of the seismic input energy is eventually dissipated via inelastic behavior in the wall. In contrast, Figure 41(b) demonstrates a significant reduction in energy dissipation demand on the wall (reduction of approximately 78% compared to no damper case) while the viscous energy dissipated by the fluid damper represents a large portion of the final seismic input energy (approximately 64%). Thus, the fluid damper has effectively provided for a transfer of energy dissipation demand from the wall to the damper. One may also note that the seismic input energy is not the same for the wall with and without the damper. This is the case since the input energy, as defined herein, is the integral of the base shear over the ground displacement. Since the base shear is different for the wall with and without the damper, the input energy is also different.

<table>
<thead>
<tr>
<th>Damage State</th>
<th>Wall Condition</th>
<th>Performance Level</th>
<th>Drift Ratio Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>No damage</td>
<td>---</td>
<td>O – Operational</td>
<td>---</td>
</tr>
<tr>
<td>Slight damage</td>
<td>Minor cracking.</td>
<td>IO – Immediate Occupancy</td>
<td>1% transient 0.25% permanent</td>
</tr>
<tr>
<td>Moderate damage</td>
<td>Large cracks at corners of door/window openings.</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Extensive damage</td>
<td>Large cracks across shearwalls.</td>
<td>LS – Life Safety</td>
<td>2% transient 1% permanent</td>
</tr>
<tr>
<td>Complete damage</td>
<td>Large permanent displacements.</td>
<td>CP – Collapse Prevention</td>
<td>3% transient or permanent</td>
</tr>
<tr>
<td>Collapse</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
</tbody>
</table>

The performance of fluid dampers for seismic energy dissipation of woodframe structures is quantified by the damping ratio from 2% to 5.7% (a 185% increase) resulting in a 28.8% reduction in peak drift while an increase in the damping ratio from 5.7% to 20.4% (a 258% increase) results in an additional 25.2% reduction in peak drift. In general, one can expect diminishing returns as the damping ratio is increased to higher levels.
Figure 39  Drift Ratio for Wall With and Without Dampers When Subjected to Taft Record: (a) Time History for No Damper; (b) Time History for C = 87.6 kN-s/m and; (c) Summary of Peak Values.
Figure 40  Base Shear Coefficient for Wall With and Without Dampers When Subjected to Taft Record: (a) Time History for No Damper; (b) Time History for $C = 87.6 \text{ kN-s/m}$ and; (c) Summary of Peak Values.
5.2.2 Near-Field Earthquake Motion (Newhall Record)

The analyses for the Newhall earthquake record were performed for the shearwall alone and for the shearwall with a fluid damper installed along the diagonal of the wall (see Figure 32). The damper was assumed to exhibit linear viscous behavior as given by Eq. (4) with a damping coefficient of 87.6 kN-s/m (500 lb-s/in). Recall from Section 5.1 that the damper with a damping coefficient of 17.5 kN-s/m (100 lb-s/in) was deemed inadequate for protecting the shearwall during strong near-field events. Note that, due to limited computer storage capacity, the response of the wall was determined for only the first 12.5 sec of the Newhall earthquake record shown in Figure 35(b). This limitation was considered to be acceptable since the strongest portion of the record occurs within the first 12.5 seconds.

The effectiveness of the damper is clearly depicted in the hysteresis loops shown in Figure 42. Note that the two hysteresis loops are not plotted to the same scale on the horizontal axis. For the wall alone (i.e., no damper), the hysteresis loop clearly demonstrates the strongly nonlinear behavior of the wall for this near-field earthquake record. In fact, the drift ratio in the wall exceeds 3% and thus the wall is regarded as having exceeded the Collapse Prevention performance level (see Table 3). In other words, the wall is regarded as having failed wherein failure occurs at approximately 5.6 seconds into the earthquake (see Figure 35(b)). For the wall with the damper, the hysteresis loop consists of a combination of damper behavior and wall behavior and thus it is not readily apparent how the wall itself (i.e., the wood framing system) performed. However, a careful examination of the two loops reveals that the peak drift was reduced to 0.37 % (a reduction of 88% from the failure drift ratio of 3%) when the damper was utilized. Thus, the dampers are effective in preventing failure and minimizing damage to the wall. The peak damper force, velocity, and stroke are 8.66 kN (1.95 k), 9.88 cm/s (3.89 in/s), and 5.99 mm (0.236 in.), respectively. As discussed in Section 8, a damper with such behavior can be readily manufactured. Furthermore, one may note the larger forces that develop within the small displacement region of the pinching zone for the case of the wall with the damper. This is the result of the damper force being proportional to velocity, thus leading to large damper forces in the region of the pinching zone where the displacements are small and the velocities are large.
The development of larger forces in the pinching region is one of the reasons that the dampers are so effective.

The effectiveness of the damper is further illustrated in Figure 43 where the hysteresis loops for the wall without and with the damper are plotted to the same scale. In addition, the Collapse Prevention (CP), Life Safety (LS), and Immediate Occupancy (IO) performance levels are indicated in Figure 43. Clearly, the wall without the damper is predicted to collapse while the wall with the damper is within the IO limit state. Note that, for the wall with the damper, the hysteresis loop consists of a combination of damper behavior and wall behavior and thus it is not readily apparent how the wall itself performed. However, as shown in Figure 44, the hysteresis loop can be decomposed into its contributions from the wall and damper, allowing one to visualize the performance of the wall itself.

![Hysteresis Loops of Shearwall Without and With Fluid Dampers Subjected to Newhall Record](image)

(a) No Damper  
(b) $C = 87.6$ kN-s/m

**Figure 42** Hysteresis Loops of Shearwall Without and With Fluid Dampers Subjected to Newhall Record *(not plotted to same scale).*

![Hysteresis Loops of Shearwall Without and With Fluid Dampers Subjected to Newhall Record](image)

(a) No Damper  
(b) $C = 87.6$ kN-s/m

**Figure 43** Hysteresis Loops of Shearwall Without and With Fluid Dampers Subjected to Newhall Record *(plotted to same scale).*
The effect of the damper on the peak drift may also be observed in the time-histories shown in Figure 45(a) and (b). The large pulse-like response that is often associated with strong, near-field ground motions causes the wall with no damper to reach its failure capacity. In contrast, the pulse-like response is completely suppressed for the wall with the damper. The peak drift is reduced by 88% (as compared to the Collapse Prevention drift ratio of 3%) when the damper is utilized. In addition, Figure 45(c) and (d) shows that the peak base shear is reduced by 36% (as compared to the Collapse Prevention base shear coefficient of 0.66) when the damper is utilized.

The performance of the fluid dampers may also be evaluated by considering the energy distribution within the wall during the earthquake. The time histories of various energy quantities are shown in Figure 46 for the wall with and without the fluid damper. Note that the two plots shown are plotted at different scales on both the horizontal and vertical axes. Figure 46(a) indicates that, without a fluid damper, essentially all of the seismic input energy up to the failure point is dissipated via inelastic behavior in the wall. In contrast, Figure 46(b) demonstrates a significant reduction in energy dissipation demand on the wall (reduction of
inelastic energy dissipation demand of approximately 90% as compared to Collapse Prevention inelastic energy dissipation demand of 2.73 kN-m). In addition, the viscous energy dissipated by the fluid damper represents a large portion of the final seismic input energy (approximately 57%). Thus, the fluid damper has effectively provided for a transfer of energy dissipation demand from the wall to the damper. One may also note that the time-history of seismic input energy is not the same for the wall with and without the damper. This is the case since the input energy, as defined herein, is the integral of the base shear over the ground displacement. Since the base shear is different for the wall with and without the damper, the input energy is also different.

Figure 45 Time-Histories of Drift and Base Shear Response for Wall With and Without Damper Subjected to Newhall Record.
Figure 46  Energy Distribution Within Wall for Newhall Record: (a) No Damper and (b) With Damper.
6. Model of Multi-Story Three-Dimensional Woodframed Building

Motivated by the encouraging results from the analysis of a single shearwall with a supplemental fluid damper, further analyses were performed on a model of a three dimensional house. The objectives of these analyses were to investigate the effectiveness of the dampers for a multi-story structure with multiple shearwalls and irregular openings. Although the detailed finite element model presented in Section 3 was able to successfully reproduce the static and dynamic behavior of a shearwall, that modeling approach was too refined to reasonably analyze an entire structure. Therefore, as described below, a simplified structural model was developed and calibrated against previous experimental and numerical results.

6.1 Description of Building

The geometry of the building model was similar to the full-scale, two-story, light-framed wood residential building recently tested as part of the CUREE-Caltech Woodframe Project by Fischer et al. (2001). The building was tested in various configurations (e.g., some with and some without finish materials) on a shaking table at the University of California at San Diego (UCSD). The plan dimensions of the UCSD building are 16 ft x 20 ft (4.88 m x 6.10 m) with nominal story heights of 9 ft (2.44 m).

For the numerical analyses described herein, the plan dimensions and story heights of the UCSD building were modified to 19 ft x 24 ft (5.79 m x 7.32 m) and 8 ft (2.4 m), respectively, to facilitate the direct use of the 8 ft x 8 ft (2.4 m x 2.4 m) shearwall model described in Section 3. Elevation and plan views of the building model are shown in Figure 47(a) and (b), respectively. In Figure 47(b), the solid rectangles indicate the location of shearwalls. Two different versions of the building were analyzed; one with symmetric placement of wall panels and the other with asymmetric placement (to simulate the effects of asymmetry due to, for example, a large garage door opening). In both cases, the numerical analyses were performed for the case of seismic excitation in the East-West direction only (see Figure 47(a)). The window and door openings were accounted for by omitting full-height wall panels at the approximate location of these openings. The effects of wall finish materials and interior partition walls were not considered. Although the building used for the numerical analyses was not meant to accurately represent the UCSD building, it may be regarded as adequate for capturing the global behavior of a building with similar characteristics to that of the UCSD building while simultaneously permitting efficient analyses.
Figure 47 Geometry of Building Model: (a) Isometric View of Both Symmetric and Asymmetric Building and (b) Plan Views of Symmetric and Asymmetric Building (Solid Rectangles Indicate Location of Shearwalls).
6.2 Finite Element Model

A nonlinear finite element model of the woodframed building was developed for the numerical analyses using the commercial program ABAQUS (ABAQUS 1998) (see Figure 48). Based on both experimental and numerical observations, the hysteretic behavior of shearwalls is primarily characterized by the behavior of the connections. In other words, the shape of the base shear versus drift hysteresis loops for a shearwall is qualitatively of the same form as the force versus deflection hysteresis loops for the connections (e.g., compare Figures 26(c) and 29(c)). Therefore, the same form of backbone curve and hysteretic algorithm used for the connection element can be used to model the shearing behavior of an entire shearwall. A simplified equivalent representation of the shearwall was thus developed as explained below and illustrated in Figure 49:

- **Studs**, which have high axial stiffness relative to their shear stiffness, are designed to carry the vertical load in the wall. Resistance to shearing is provided by the sheathing panels. Without sheathing panels, the studs, sill plate, and sole plate form a mechanism, essentially connected with hinges. Therefore, truss elements were used to form the framework of the simplified equivalent shearwall. The two vertical elements of the framework were assigned an axial stiffness equal to that of half of the studs in the detailed shearwall model.

- **Shear resistance** of the simplified equivalent shearwall is provided by two diagonal bracing elements. The hysteretic behavior of the bracing elements was based upon the connection element of the detailed wall model. The values of the parameters that define the hysteretic behavior of the simplified shearwall were determined via a calibration procedure in which the response of both the simplified wall and the detailed wall model were matched when subjected to both static and dynamic loading. A comparison between the response of the simplified wall model and the detailed wall model is presented in Figure 50 for static pushover analysis and Figure 51 for seismic analysis. The seismic analysis was performed using the first 9 seconds of the Taft record as input (see Figure 35(a)). For the static loading, the comparison is quite good. For the dynamic loading, the shape of the hysteresis loops are qualitatively of the same form but with peak forces and displacements that do not match (12.4% error in peak force and 23.0% error in peak displacement). The difference in the dynamic response is also depicted in the drift and base shear time histories shown in Figure 52. The time histories indicate that the two responses compare quite well in terms of frequency content but exhibit the aforementioned differences in peak response. Although the simplified model is not an accurate representation of the detailed shearwall model, it is deemed adequate for the purpose of evaluating the effect of fluid dampers on the seismic response of the building model.

- The roof and floor diaphragms were modeled using 4-node, isoparametric shell elements. Stiffness properties were based upon typical housing construction, with the intention of producing diaphragms that are very stiff in plane relative to the in-plane stiffness of the shearwalls.

- The mass of the roof and floor diaphragms was accounted for via a consistent mass matrix in which the assigned weight density was typical of the weight for design of floors and roofs (Ambrose and Vergun 1987). The contribution to mass from the walls was lumped to the
nodes at the perimeter of the diaphragms on the basis of tributary area and typical weight values for walls. The resulting total mass of the symmetric building was 8,344 kg (47.6 lb-s^2/in) with 3,353 kg (19.13 lb-s^2/in) being assigned to the roof level, 3,811 kg (21.74 lb-s^2/in) assigned to the floor level, and the remaining 1,180 kg (6.73 lb-s^2/in) assigned to the ground level. The mass associated with the additional shearwall in the first story of the asymmetric building (3.5% of the total weight of the symmetric building) was regarded as insignificant and thus was not accounted for in the mass of the asymmetric building.

Figure 48 Finite Element Model of Woodframed Building (Shear Panels Not Shown).

(a) (b)

Figure 49 Finite Element Models of: (a) Detailed Shearwall and (b) Simplified Shearwall.
Figure 50  Comparison of Static Pushover Response for Detailed Wall Model and Simplified Wall Model.

Figure 51  Comparison of Response for Detailed Wall Model and Simplified Wall Model When Subjected to Taft Record.
Figure 52  Comparison of Time-Histories of Response for Detailed Wall Model and Simplified Wall Model When Subjected to Taft Record: a) Drift Ratio and b) Base Shear Coefficient.
7. Effect of Fluid Damper on Woodframed Building Behavior

7.1 Dynamic Properties of Building

The dynamic properties of the building were determined via an undamped eigenvalue analysis using the initial elastic stiffness properties of the building model. For the symmetric building, the natural frequency in the first three modes of vibration are 4.38 Hz (East-West translation), 6.21 Hz (North-South translation), and 8.29 Hz (torsion) with the corresponding mode shapes shown in Figure 53. For the asymmetric building, the natural frequency in the first three modes of vibration are 5.03 Hz (East-West translation), 6.21 Hz (North-South translation), and 8.71 Hz (torsion) with mode shapes essentially identical to those shown in Figure 53. The inherent damping in the building was accounted for via a Rayleigh damping formulation wherein, for low amplitudes of vibration, the damping ratio in the first and second modes of vibration were assumed to be 2% and 10%, respectively.

![Mode 1 (EW Translation)](image1)
![Mode 2 (NS Translation)](image2)
![Mode 3 (Torsion)](image3)

**Figure 53** First Three Mode Shapes for Symmetric Building.
7.2 Seismic Response

Both the symmetric and asymmetric building were subjected to seismic excitation in the East-West direction (see Figure 47(a)) using the earthquake excitations described in Section 5.2. For the case of the building with dampers, dampers having a damping coefficient of \( C = 87.6 \text{kN-s/m} \) were located along the diagonal of all of the shearwalls of the first story (see Figure 47(b) for location of first story shearwalls). Thus, there are six dampers within the first story of the symmetric building (two on each of the East and West walls and one on each of the North and South walls) and seven dampers within the first story of the asymmetric building (two on each of the East and West walls, one on the North wall, and two on the South wall). No dampers were located in the second story since preliminary analyses revealed that the effects of the dampers were most significant when the dampers were located in the first story. Note that, for the asymmetric building, no attempt was made to distribute the dampers so as to control the torsional response. Due to limited computer storage capacity, the response of the building subjected to both the Taft and Newhall earthquake records was determined for only the first 15 seconds of the records shown in Figure 35. This limitation was considered to be acceptable since the strongest portion of the two records occur within the first 15 seconds.

7.2.1 Symmetric Building – Taft Earthquake Record

The hysteresis loops for the first and second stories are shown in Figure 54 for the symmetric building with no dampers. The hysteresis loops are plotted to the same scale to allow for a qualitative comparison of the first and second story response. Note that the second story shear is normalized with respect to the total weight of the building. Based on the damage levels described in Table 3, it is evident that the first story and second story would likely perform at an Operational to Immediate Occupancy performance level with essentially no damage (peak drift ratio is approximately 0.25%). Any damage that might occur would likely be reduced if the effects of wall finish materials and interior partitions were considered. Since there is very little damage to the symmetric building for the Taft record, an analysis of the response with dampers included in the building is not presented herein.

![Hysteresis Loops for Symmetric Building Without Dampers Subjected to Taft Record.](image)

**Figure 54** Hysteresis Loops for Symmetric Building Without Dampers Subjected to Taft Record.
7.2.2 Symmetric Building – Newhall Earthquake Record

The hysteresis loops for the first and second stories are shown in Figure 55 for the symmetric building with no dampers. Note that the two hysteresis loops are not plotted to the same scale and that only the portion of the predicted response prior to failure is shown. In this case, failure occurs at approximately 5.6 seconds into the earthquake (see Figure 35(b)). Based on the damage levels described in Table 3 and indicated in Figure 55, it is evident that the first story would likely collapse (peak drift ratio exceeds 3%) while at the same time the second story would have experienced minor damage (peak drift ratio less than 1%). It should be noted that collapse may not be predicted for the building if the effects of wall finish materials and interior partitions are considered.

![First Story Hysteresis Loop](image1)

![Second Story Hysteresis Loop](image2)

**Figure 55** Hysteresis Loops for Symmetric Building Without Dampers Subjected to Newhall Record.

For the near-field Newhall record, the hysteresis loops for the first and second stories are shown in Figure 56 for the building with dampers. Note that the two hysteresis loops are not plotted to the same scale. The effect of the dampers is to increase the peak base shear in the first story by 46%. This is not surprising since the dampers introduce additional forces which must be transferred through the lateral force resisting system. In addition, the inclusion of the dampers reduced the peak drift ratio by 58% in comparison to the Collapse Prevention drift ratio of 3%. For the building with dampers, the peak force, velocity and stroke in a single damper was 25.3 kN (5.69 kips), 28.9 cm/s (11.4 in/s), and 2.14 cm (0.844 in), respectively. Based on the damage levels described in Table 3, it is evident from Figure 56 that the first story would experience slight to moderate structural damage while the second story would experience essentially no damage. If the effects of finish materials and interior partitions were considered, even less damage would be expected.
The performance of the fluid dampers can also be evaluated via examination of the time-dependent energy distribution within the building during the earthquake. The time histories of various energy quantities are shown in Figure 57 for the building with and without the fluid dampers. For the building without dampers, only the portion of the energy distribution prior to failure is shown. At failure, 83% of the input energy has been absorbed by inelastic behavior. In contrast, for the building with dampers, at the end of the analysis the inelastic strain energy demand is 44% of the input energy while the viscous energy accounts for 55% of the input energy. Thus, the viscous energy dissipated by the fluid damper represents a large portion of the final seismic input energy. The fluid damper has effectively provided for a transfer of energy dissipation demand from the building to the damper.

**Figure 56** Hysteresis Loops for Symmetric Building With Dampers Subjected to Newhall Record.

**Figure 57** Energy Distribution Within Symmetric Building Subjected to Newhall Record.
7.2.3 Asymmetric Building – Taft Earthquake Record

The asymmetric building is prone to twisting about a vertical axis. Thus, the shear forces and interstory drifts can vary for each shearwall within the building. To evaluate the response of the building with and without dampers, the hysteresis loops of the first story walls that resist the largest shear forces and drifts are compared. The response of the first story is emphasized since, as expected, the symmetric analyses demonstrated that the first story response tends to be much stronger than the second story. Also, it should be clear that when a wall is referred to, the intent is to include all individual wall elements within the wall [e.g., the first story North wall contains one wall element while the first story South wall contains two wall elements (see Figure 47(b))].

For the far-field Taft record, the hysteresis loops for the first story North and South walls of the building with and without dampers are shown in Figure 58. For the purpose of qualitative comparison, the hysteresis loops are shown to the same scale. The hysteresis loops for the East and West walls are not shown since these walls experience very minor deformation (peak drift ratio of 0.026% with no dampers). The Wall Shear Coefficient on the vertical axis is equal to the wall shear force normalized by the total weight of the building while the Wall Drift Ratio on the horizontal axis is equal to the wall interstory drift normalized by the wall height. Note that, for building without dampers, the peak shear force in the South wall is approximately twice that of the North wall since the South wall contains twice as many shearwalls (see Figure 47(b)). Based on the damage levels described in Table 3, it is evident from Figure 58 that the asymmetric building subjected to the Taft earthquake record would experience minor damage both with and without fluid dampers. Any damage that might occur would likely be reduced if the effects of wall finish materials and interior partitions were considered. Since there is very little damage to the asymmetric building for the Taft earthquake record, no further analysis was performed for this case.
As for the Taft earthquake record, the response of the first story is emphasized since, as expected, the symmetric analyses demonstrated that the first story response tends to be much stronger than the second story. For the asymmetric building without dampers and subjected to the near-field Newhall record, the hysteresis loops for each wall of the first story are shown in Figure 59. Note that the North and South wall are plotted to the same scale. Similarly for the East and West walls. It is clear from Figure 59 that the North and South walls primarily resist the earthquake loading. Furthermore, both the North and South walls have exceeded the Life-Safety performance level and are approaching the Collapse Prevention performance level. However, the damage to the building would likely be reduced if the effects of wall finish materials and interior partitions were considered. Note that the peak shear force in the South wall is approximately twice that of the North wall since the South wall contains twice as many shearwalls (see Figure 47(b)).
For the asymmetric building with dampers and subjected to the near-field Newhall record, the hysteresis loops for each wall of the first story are shown in Figure 60. Note that the North and South wall are plotted to the same scale. Similarly for the East and West walls. It is clear from Figure 60 that the North and South walls primarily resist the earthquake loading. Furthermore, note that the peak shear force in the South wall is approximately twice that of the North wall since the South wall contains twice as many shearwalls (see Figure 47(b)). The effect of the dampers is to increase the peak shear force in the North and South walls by 20.1% and 5.5%, respectively, and to reduce the peak drift ratio in the North and South Wall by 67.8% and 68.0%, respectively, as compared to the asymmetric building without dampers. Evidently, the dampers are very effective for controlling the building deformations but are not effective in controlling the building shear forces. This is to be expected since the building behaves as a nonlinear system in which the ultimate force in the wall framing system is limited (see Figure 28) while the dampers introduce additional forces which must be transferred through the building into the foundation. The peak force, velocity and stroke for a single damper was 19.0 kN (4.26 kips), 21.6 cm/s (8.5 in/s), and 1.42 cm (0.561 in), respectively. Based on Table 3 and examination of Figure 60, the North and South walls are below the Immediate Occupancy performance level. In
summary, the dampers are very effective in minimizing damage to the asymmetric building when it is subjected to the strong near-field Newhall earthquake record.

Figure 60  Hysteresis Loops for First Story of Asymmetric Building With Dampers Subjected to Newhall Record.

The torsional response of the asymmetric building subjected to the Newhall earthquake record is evident in Figures 59 and 60 where the response of the North and South wall is not identical. Similarly for the response of the East and West wall of Figures 59 and 60. The torsional response may also be directly observed by evaluating the rotation of the building. For example, the rotation of the second story floor level with and without dampers is shown in Figure 61. The torsional response for the building with dampers is reduced by 68% as compared to the building without dampers in spite of the fact that the dampers were not distributed within the building in an effort to control the torsional response. In general, the distribution of fluid dampers may be selected such that torsional response is inhibited (Goel 1998).
The performance of the fluid dampers may also be evaluated by considering the energy distribution within the asymmetric building during the earthquake. The time histories of various energy quantities are shown in Figure 62 for the building with and without fluid dampers. Note that the two plots shown are plotted at different scales on the vertical axis. Figure 62(a) indicates that, without fluid dampers, essentially all of the seismic input energy is eventually dissipated via inelastic behavior in the wall framing system. In contrast, Figure 62(b) demonstrates a significant reduction in energy dissipation demand on the wall framing system (reduction of 63% compared to no damper case) while the viscous energy dissipated by the fluid dampers represents a significant portion (56%) of the final seismic input energy. Thus, the fluid dampers effectively provided for a transfer of energy dissipation demand from the building to the dampers. One may also note that the seismic input energy is not the same for the building with and without the dampers. This is the case since the input energy, as defined herein, is the integral of the base shear over the ground displacement. Since the base shear is different for the building with and without the dampers, the input energy is also different.
8. Implementation Issues and Recommendations for Further Study

For the shearwall and house model analyzed herein, the peak force, velocity, and stroke [25.3 kN (5.7 kips), 28.9 cm/s (11.4 in/s), and 2.14 cm (0.844 in), respectively] for a single damper occurred for the symmetric building model subjected to the Newhall earthquake record. These peak values define the capacity requirements for a damper having a damping coefficient of 87.6 kN-s/m. According to a prominent designer/manufacturer of viscous fluid dampers, dampers with such characteristics can be readily manufactured. However, a number of issues remain to be addressed before fluid dampers will find implementation in light-framed wood buildings.

Prior to implementation, experimental testing needs to be performed to validate the general results presented herein. In particular, testing should be performed on full-scale woodframed shearwalls and three-dimensional building systems with and without both linear and nonlinear fluid dampers. Experimental tests are critically important for identifying sources of error in mathematical and computational models utilized in numerical simulations.

It is recognized that typical light woodframed structures are primarily constructed in the field. To aid in the installation of fluid dampers in such structures, it is likely that the damper would need to be installed within a pre-fabricated modular wall unit that can be installed in strategic locations in a woodframe house in order to act as the primary lateral load-resisting system (similar to the recently developed Simpson Strong-Tie Strong-Wall® Shearwall). The modular wall units would be constructed in a controlled manufacturing environment with damper connections that produce essentially no slip prior to damper engagement. Minimizing slip will be important for controlling the level of damage during earthquakes; particularly for frequent, weak earthquakes that, while producing damage in the structure, generate relatively small wall displacements. In addition, construction tolerances may be such that, prior to damper engagement, the building deforms sufficiently to cause wall finish damage. Note that the use of modular wall units would result in changes in the load path and the torsional resistance of the building, leading to a potentially more complex design and analysis process for both the wood framing system and the fluid dampers.

Although a rigorous evaluation of the optimal damper locations (both plan-wise and elevation-wise) within a general woodframed structure was not performed, the seismic analysis performed herein on the two-story building model indicated that the dampers were more beneficial when located in the first story rather than the second. This may be attributed to the larger mass that drives the first story (i.e., first floor plus roof) as compared to the mass which drives the second story (i.e., roof only). Furthermore, the use of dampers within the first story only appeared to be adequate for the particular structure analyzed. For a typical two-story residential building with a floor area of, say, 92.9 m² per floor (1000 ft²) and four external walls, it is expected that one or two dampers per external first story wall would provide sufficient supplemental damping to significantly reduce the inelastic response of the building when subjected to strong earthquakes. If the damper wall units were to be mass-produced, a conservative estimate of the cost of each damper within a wall unit would be approximately $600 based on consultation with a fluid damper manufacturer. Thus, for one or two dampers per external wall, the total additional cost for the supplemental dampers would be approximately $2,400 to $4,800 (assuming that the pre-fabricated wood framing has the same cost as the field-fabricated framing).
Although not explored in this study, recently developed amplification systems for stiff structures (Constantinou et al., 2001) may be well-suited to light-framed wood buildings which experience inelastic deformations at relatively low displacements. Such amplification systems utilize the geometrical configuration of the damper within the wall to amplify the effects of the damper.

In summary, to move toward implementation of fluid dampers for seismic protection of wood-framed buildings, it is suggested that the priorities for future research studies should include:

- A comprehensive experimental study in which tests are performed on full-scale wood-framed shearwalls and simple three-dimensional building systems with and without linear and nonlinear fluid dampers. The parameters controlling the numerical simulations in the study contained herein would then be modified to account for the observed experimental behavior of the combined damper/shearwall and damper/building systems.

- Consideration of the effects of construction tolerances on the ability of the damper to achieve high levels of performance such as Immediate Occupancy. This issue could be realistically examined via the experimental testing described above.

- An evaluation of the effects of wall finish materials and interior partitions on the dynamic response of shearwalls and buildings with fluid dampers. These issues could be realistically examined via the experimental testing described above.

- An evaluation of the effects of damper installation on load paths and the optimal distribution of dampers for torsional resistance (plan-wise distribution).

- Assuming successful experimental testing, simplified design and analysis procedures for wood-framed buildings that incorporate fluid dampers would need to be developed to encourage utilization of supplemental fluid damper systems. This is particularly important for light-framed residential construction in which the design is generally not based on fundamental engineering principles but rather is largely based on historical conventions and rules-of-thumb.

- Cost-benefit analyses need to be performed to evaluate the economic implications of implementing a fluid damper protection system. A suitable starting point for such an analysis would include the relationships developed in Task 1.4.6 of the Woodframe Project (McMullin and Merrick 2002) between drift ratios and cost of finish material damage.
9. Summary and Conclusions

Although there has been a steadily increasing growth in the application of advanced seismic protection systems (i.e., base isolation and supplemental damping systems) within concrete and steel structures, the applications within wood structures is essentially non-existent. This is in spite of the fact that light-framed wood structures have experienced extensive damage during previous strong earthquakes (e.g., the 1994 Northridge Earthquake).

The numerical analyses presented herein represents the first study on the application of fluid dampers to light-framed wood buildings. Fluid dampers are capable of providing a reliable, non-degrading source of energy dissipation for woodframed buildings. Furthermore, due to their high energy dissipation density, fluid dampers are well-suited to provide superior protection of structures subjected to strong earthquakes and thus of meeting stringent requirements of performance-based earthquake engineering specifications. The results of seismic analyses for a single shearwall and a two-story light-framed building offer convincing evidence of the potential benefit of fluid dampers for seismic protection of light-framed wood structures. For small magnitude, far-field, weak earthquakes, the application of fluid dampers resulted in minimal damage to the wood-framed building, allowing it to continue to perform well during aftershocks or future large magnitude earthquakes. For large magnitude, near-field, strong earthquakes, the application of fluid dampers was shown to prevent collapse of the woodframed building by dissipating a large percentage of the seismic input energy, relieving the inelastic energy dissipation demand on the structure.

Issues that remain to be addressed prior to implementation of fluid dampers in light-framed wood structures include the need for experimental testing, the consideration of the effects of construction tolerances, an evaluation of the effects of wall finish materials and interior partitions, an evaluation of the effects of damper installation on load paths, an evaluation of the optimal distribution of dampers for torsional resistance, the need for simplified design and analysis procedures, and the need for a cost-benefit analyses.
10. References


