Experimental and Analytical Investigation of Seismic Retrofit of Structures with Supplemental Damping: Part II - Friction Devices

by

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Technical Report NCEER-95-0009

July 6, 1995

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This is the second in a series of NCEER technical reports by the authors addressing capabilities and limitations of passive energy dissipation systems through performance comparative studies. Friction devices are considered in this report through a combined experimental and analytical study. The 1:3 scale reinforced concrete frame, the same one used in the first study, was again used for experimental verification. The results show that the retrofit of reinforced concrete structures with friction damping devices can produce satisfactory seismic response. The damping enhancement contributes to the reduction of maximum deformations and only slightly modifies the structural forces transmitted to the foundation.

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PREFACE

The National Center for Earthquake Engineering Research (NCEER) was established to expand and disseminate knowledge about earthquakes, improve earthquake-resistant design, and implement seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures in the eastern and central United States and lifelines throughout the country that are found in zones of low, moderate, and high seismicity.

NCEER’s research and implementation plan in years six through ten (1991-1996) comprises four interlocked elements, as shown in the figure below. Element I, Basic Research, is carried out to support projects in the Applied Research area. Element II, Applied Research, is the major focus of work for years six through ten. Element III, Demonstration Projects, have been planned to support Applied Research projects, and will be either case studies or regional studies. Element IV, Implementation, will result from activity in the four Applied Research projects, and from Demonstration Projects.

Research in the Building Project focuses on the evaluation and retrofit of buildings in regions of moderate seismicity. Emphasis is on lightly reinforced concrete buildings, steel semi-rigid frames, and masonry walls or infills. The research involves small- and medium-scale shake table tests and full-scale component tests at several institutions. In a parallel effort, analytical models and computer programs are being developed to aid in the prediction of the response of these buildings to various types of ground motion.
Two of the short-term products of the Building Project will be a monograph on the evaluation of lightly reinforced concrete buildings and a state-of-the-art report on unreinforced masonry.

The protective and intelligent systems program constitutes one of the important areas of research in the Building Project. Current tasks include the following:

1. Evaluate the performance of full-scale active bracing and active mass dampers already in place in terms of performance, power requirements, maintenance, reliability and cost.
2. Compare passive and active control strategies in terms of structural type, degree of effectiveness, cost and long-term reliability.
3. Perform fundamental studies of hybrid control.
4. Develop and test hybrid control systems.

This is the second in a series of NCEER technical reports by the authors addressing capabilities and limitations of passive energy dissipation systems through performance comparative studies. Friction devices are considered in this report through a combined experimental and analytical study. The 1:3 scale reinforced concrete frame, the same one used in the first study, was again used for experimental verification. The results show that the retrofit of reinforced concrete structures with friction damping devices can produce satisfactory seismic response. The damping enhancement contributes to the reduction of maximum deformations and only slightly modifies the structural forces transmitted to the foundation.
ABSTRACT

The need for structures which function more reliably without damage during severe earthquakes was reemphasized by the behavior of structures during recent earthquakes (Loma Prieta 1989, Northridge 1994, Kobe 1995, etc). The existing structures and often new ones must rely on large inelastic deformations in hysteretic behavior to dissipate the motion's energy, while the capacity to sustain such deformations is limited by previous non-ductile design or limitations of materials. An alternative method to reduce the demand of energy dissipation in the gravity load carrying elements of structures is the addition of damping devices. These devices dissipate most energy through heat transfer and reduce the deformation demands. In inelastic structures the supplemental damping mechanism reduces primarily deformations with small changes in the strength demand. The main benefit of added damping in the inelastic structures is the reduction of the demand for energy dissipation in the gravity load carrying structural members, thus reducing the deterioration of their low cycle fatigue capacity.

An experimental investigation of different damping devices was carried out individually to allow for physical and mathematical modeling of their behavior. A series of shaking table tests of a 1:3 scale reinforced concrete frame incorporating these devices were performed after the frame was damaged by prior severe (simulated) earthquakes.

Several different damping devices were used in this study: (a) viscoelastic, (b) fluid viscous, (c) friction (of two types) and (d) fluid viscous walls. An analytical platform for evaluation of structures integrating such devices was developed and incorporated in IDARC Version 3.2 (Kunnath and Reinhorn, 1994). The experimental and analytical
study shows that the dampers can reduce inelastic deformation demands and, moreover, reduce the damage, quantified by an index monitoring permanent deformations. The structures with friction dampers are able to shift their structural frequencies and increase the energy dissipation with the increase of earthquake intensity to survive a strong earthquake. The general structure's force response is mostly reduced or minimally increased due to the effects of both damping and stiffening. An evaluation of efficiency of dampers using a simplified pushover analysis method was investigated as an alternative method for prediction of structural behavior and design.

This report, second in a series, presents the evaluation of friction dampers used as additional braces in reinforced concrete frame structures.
ACKNOWLEDGMENTS

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# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>SEC.</th>
<th>TITLE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>INTRODUCTION</td>
<td>1-1</td>
</tr>
<tr>
<td>1.1</td>
<td>Viscoelastic Devices</td>
<td>1-4</td>
</tr>
<tr>
<td>1.2</td>
<td>Viscous Walls</td>
<td>1-7</td>
</tr>
<tr>
<td>1.3</td>
<td>Fluid Viscous Dampers</td>
<td>1-9</td>
</tr>
<tr>
<td>1.4</td>
<td>Hysteresis Devices</td>
<td>1-12</td>
</tr>
<tr>
<td>1.4.1</td>
<td>Friction Devices</td>
<td>1-12</td>
</tr>
<tr>
<td>1.4.2</td>
<td>Elastomeric Spring Dampers</td>
<td>1-18</td>
</tr>
<tr>
<td>1.4.3</td>
<td>Metallic Systems</td>
<td>1-20</td>
</tr>
<tr>
<td>1.4.3.1</td>
<td>Yielding Steel Elements</td>
<td>1-20</td>
</tr>
<tr>
<td>1.4.3.2</td>
<td>Lead Extrusion Devices (LEDs)</td>
<td>1-25</td>
</tr>
<tr>
<td>1.4.3.3</td>
<td>Shape Memory Alloys (SMAs)</td>
<td>1-17</td>
</tr>
<tr>
<td>1.4.3.4</td>
<td>Eccentrically Braced Frame (EBF)</td>
<td>1-28</td>
</tr>
<tr>
<td>1.5</td>
<td>Code Provision for Design of Structures Incorporating Passive Energy Dissipating Devices</td>
<td>1-31</td>
</tr>
<tr>
<td>1.6</td>
<td>Objectives of This Investigation</td>
<td>1-31</td>
</tr>
<tr>
<td>2.</td>
<td>FRICTION DAMPERS</td>
<td>2-1</td>
</tr>
<tr>
<td>2.1</td>
<td>Description of Tekton Friction Damping Devices</td>
<td>2-1</td>
</tr>
<tr>
<td>2.2</td>
<td>Description of Sumitomo Friction Damping Devices</td>
<td>2-1</td>
</tr>
<tr>
<td>2.3</td>
<td>Operation of Dampers</td>
<td>2-4</td>
</tr>
<tr>
<td>2.4</td>
<td>Testing of Damping Devices</td>
<td>2-4</td>
</tr>
<tr>
<td>3.</td>
<td>ANALYTICAL MODELING OF FRICTION DAMPERS</td>
<td>3-1</td>
</tr>
<tr>
<td>3.1</td>
<td>Mathematical Modeling</td>
<td>3-1</td>
</tr>
<tr>
<td>3.1.1</td>
<td>Bouc-Wen's Model</td>
<td>3-1</td>
</tr>
<tr>
<td>3.1.2</td>
<td>Coulomb Friction-Viscous Damping Model</td>
<td>3-2</td>
</tr>
<tr>
<td>3.1.3</td>
<td>Modeling of Tested Dampers</td>
<td>3-5</td>
</tr>
<tr>
<td>4.</td>
<td>EXPERIMENTAL STUDY OF RETROFITTED STRUCTURE - EARTHQUAKE SIMULATOR TESTING</td>
<td>4-1</td>
</tr>
<tr>
<td>4.1</td>
<td>Retrofit of Damaged Reinforced Concrete Model</td>
<td>4-1</td>
</tr>
<tr>
<td>4.2</td>
<td>Structure Model for Shaking Table Study</td>
<td>4-2</td>
</tr>
<tr>
<td>4.3</td>
<td>Retrofit with Supplemental Friction Dampers</td>
<td>4-8</td>
</tr>
<tr>
<td>4.3.1</td>
<td>Friction Damper</td>
<td>4-14</td>
</tr>
<tr>
<td>4.4</td>
<td>Instrumentation</td>
<td>4-15</td>
</tr>
<tr>
<td>4.5</td>
<td>Experimental Program</td>
<td>4-19</td>
</tr>
<tr>
<td>4.6</td>
<td>Identification of Structure Properties</td>
<td>4-22</td>
</tr>
<tr>
<td>4.6.1</td>
<td>Experimental Identification of Dynamic Characteristics of Model</td>
<td>4-22</td>
</tr>
<tr>
<td>4.6.2</td>
<td>Dynamic Characteristics of Structure</td>
<td>4-36</td>
</tr>
<tr>
<td>4.6.2.1</td>
<td>Structure without Supplementary Dampers</td>
<td>4-36</td>
</tr>
<tr>
<td>SEC.</td>
<td>TITLE</td>
<td>PAGE</td>
</tr>
<tr>
<td>------</td>
<td>-----------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>4.6.2.2</td>
<td>Structure with Supplementary Dampers</td>
<td>4-38</td>
</tr>
<tr>
<td>4.7</td>
<td>Seismic Response</td>
<td>4-45</td>
</tr>
<tr>
<td>4.8</td>
<td>Summary of Experimental Study</td>
<td>4-80</td>
</tr>
<tr>
<td>5</td>
<td>MODELING OF INELASTIC STRUCTURE WITH SUPPLEMENTAL DAMPERS</td>
<td></td>
</tr>
<tr>
<td>5.1</td>
<td>Modeling of Inelastic Structure</td>
<td>5-1</td>
</tr>
<tr>
<td>5.2</td>
<td>Modeling of Structure with Supplemental Dampers</td>
<td></td>
</tr>
<tr>
<td>5.2.1</td>
<td>Modeling Using Bouc-Wen's Model</td>
<td>5-2</td>
</tr>
<tr>
<td>5.2.2</td>
<td>Solution of Differential Equations</td>
<td>5-5</td>
</tr>
<tr>
<td>5.2.3</td>
<td>Solution of Seismic Response of Structure</td>
<td>5-7</td>
</tr>
<tr>
<td>5.2.4</td>
<td>Analytical Damage Evaluation</td>
<td>5-7</td>
</tr>
<tr>
<td>5.2.5</td>
<td>Determining the Monotonic Strength Envelope</td>
<td>5-9</td>
</tr>
<tr>
<td>5.2.6</td>
<td>Monotonic Strength Envelope with Braces</td>
<td>5-11</td>
</tr>
<tr>
<td>5.3</td>
<td>Validation of Structural Model with Friction Dampers</td>
<td>5-11</td>
</tr>
<tr>
<td>5.3.1</td>
<td>Time History Analysis</td>
<td>5-12</td>
</tr>
<tr>
<td>5.3.2</td>
<td>Monotonic Pushover Analysis</td>
<td>5-13</td>
</tr>
<tr>
<td>6</td>
<td>SIMPLIFIED EVALUATION OF INELASTIC RESPONSE WITH SUPPLEMENTAL DAMPING</td>
<td></td>
</tr>
<tr>
<td>6.1</td>
<td>Response Spectra for Elastic Systems</td>
<td>6-1</td>
</tr>
<tr>
<td>6.1.1</td>
<td>Composite Response Spectra for Single Degree of Freedom (SDOF)</td>
<td>6-1</td>
</tr>
<tr>
<td>6.1.2</td>
<td>Composite Spectra for Multi-Degree of Freedom (MDOF)</td>
<td>6-2</td>
</tr>
<tr>
<td>6.1.2.1</td>
<td>Composite Spectra for a Single Mode</td>
<td>6-4</td>
</tr>
<tr>
<td>6.1.2.2</td>
<td>Composite Spectra Including Higher Modes</td>
<td>6-4</td>
</tr>
<tr>
<td>6.2</td>
<td>Evaluation of Seismic Demand in Elastic Structures</td>
<td>6-6</td>
</tr>
<tr>
<td>6.2.1</td>
<td>Response without Supplemental Damping</td>
<td>6-6</td>
</tr>
<tr>
<td>6.2.2</td>
<td>Response with Supplemental Damping</td>
<td>6-8</td>
</tr>
<tr>
<td>6.3</td>
<td>Evaluation of Motion of an Inelastic Structures</td>
<td>6-10</td>
</tr>
<tr>
<td>6.3.1</td>
<td>Response Neglecting Hysteretic Damping</td>
<td>6-11</td>
</tr>
<tr>
<td>6.3.2</td>
<td>Response Considering the Hysteretic Damping</td>
<td>6-11</td>
</tr>
<tr>
<td>6.3.2.1</td>
<td>Estimate of Equivalent Hysteretic Damping</td>
<td>6-14</td>
</tr>
<tr>
<td>6.4</td>
<td>Evaluation of Response of Inelastic Structure with Supplemental Damping</td>
<td>6-17</td>
</tr>
<tr>
<td>6.4.1</td>
<td>Influence of Damping Increase</td>
<td>6-17</td>
</tr>
<tr>
<td>6.4.2</td>
<td>Influence of Stiffening due to Supplemental Dampers</td>
<td>6-18</td>
</tr>
<tr>
<td>6.4.3</td>
<td>Influence of Dynamic Strength</td>
<td>6-18</td>
</tr>
<tr>
<td>6.5</td>
<td>Evaluation of Experimental Response (Summary)</td>
<td>6-23</td>
</tr>
<tr>
<td>7</td>
<td>CONCLUSIONS</td>
<td>7-1</td>
</tr>
<tr>
<td>8</td>
<td>REFERENCES</td>
<td>8-1</td>
</tr>
</tbody>
</table>
### APPENDIX A

<table>
<thead>
<tr>
<th>SEC.</th>
<th>TITLE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>APPENDIX A</td>
<td>A-1</td>
</tr>
<tr>
<td></td>
<td>A1.1 Reinforcement Details</td>
<td>A-1</td>
</tr>
<tr>
<td></td>
<td>A1.2 Model Materials</td>
<td>A-1</td>
</tr>
<tr>
<td></td>
<td>A1.3 Scale Factors for the Model</td>
<td>A-9</td>
</tr>
</tbody>
</table>

### APPENDIX B: INSTRUMENTATION

<table>
<thead>
<tr>
<th>SEC.</th>
<th>TITLE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>Load Cells</td>
<td>B-1</td>
</tr>
<tr>
<td>B-2</td>
<td>Displacement Transducers</td>
<td>B-1</td>
</tr>
<tr>
<td>B-3</td>
<td>Accelerometers</td>
<td>B-2</td>
</tr>
</tbody>
</table>
# LIST OF ILLUSTRATIONS

<table>
<thead>
<tr>
<th>FIG.</th>
<th>TITLE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1</td>
<td>Detail of Beam to Column Connection with Viscoelastic Material</td>
<td>1-5</td>
</tr>
<tr>
<td>1-2</td>
<td>Viscoelastic Dampers and Installation Detail</td>
<td>1-6</td>
</tr>
<tr>
<td>1-3</td>
<td>Viscous Wall, Installation Detail and Hysteresis Loops (from Miyazaki 1992)</td>
<td>1-8</td>
</tr>
<tr>
<td>1-4</td>
<td>Fluid Viscous Damper Construction and Installation Detail</td>
<td>1-10</td>
</tr>
<tr>
<td>1-5</td>
<td>Typical Hysteresis Loops of Fluid Viscous Damper</td>
<td>1-11</td>
</tr>
<tr>
<td>1-6</td>
<td>Paul Friction Damper, Typical Hysteresis Loop and Application (from Pall 1993)</td>
<td>1-14</td>
</tr>
<tr>
<td>1-7</td>
<td>Detail of Displacement Controller (from Constantinou et al 1991)</td>
<td>1-16</td>
</tr>
<tr>
<td>1-8</td>
<td>Details of SBCs and Hysteresis Loops</td>
<td>1-17</td>
</tr>
<tr>
<td>1-9</td>
<td>Energy Dissipation Restraint and Representative Force-Displacement Loops (from Nims 1993)</td>
<td>1-19</td>
</tr>
<tr>
<td>1-10</td>
<td>Elastomeric Spring Damper and Hysteresis Behavior</td>
<td>1-21</td>
</tr>
<tr>
<td>1-11</td>
<td>Details of a Yielding Steel Bracing System in a Building in New Zealand (from Tyler 1985)</td>
<td>1-22</td>
</tr>
<tr>
<td>1-12</td>
<td>ADAS Device Hysteresis Loops (from Whittaker 1991)</td>
<td>1-26</td>
</tr>
<tr>
<td>1-13</td>
<td>T-ADAS Device Hysteresis Loops (from Tsai 1992)</td>
<td>1-26</td>
</tr>
<tr>
<td>1-14</td>
<td>Lead Joint Damper and Hysteresis Loops (from Sakurai 1992)</td>
<td>1-26</td>
</tr>
<tr>
<td>1-15</td>
<td>LED Hysteresis Loops (from Robinson 1987)</td>
<td>1-29</td>
</tr>
<tr>
<td>1-16</td>
<td>SMA Superelastic Hysteresis Behavior (from Aiken 1992)</td>
<td>1-29</td>
</tr>
<tr>
<td>1-17</td>
<td>NiTi (Tension) and Cu-Zn-Al (Tension) Hysteresis Loops (from Aiken 1992, Witting 1992)</td>
<td>1-29</td>
</tr>
<tr>
<td>1-18</td>
<td>Different Kind of Eccentrically Braced Element</td>
<td>1-30</td>
</tr>
<tr>
<td>2-1</td>
<td>Construction of Tekton Friction Damper</td>
<td>2-2</td>
</tr>
<tr>
<td>2-2</td>
<td>Sectional Views of a Sumitomo Friction Damper (from Aiken 1991)</td>
<td>2-3</td>
</tr>
<tr>
<td>2-3</td>
<td>Test Results of Tekton Friction Damper for Various Frequencies</td>
<td>2-6</td>
</tr>
<tr>
<td>2-4</td>
<td>Test Results of Sumitomo Friction Damper for Various Frequencies</td>
<td>2-7</td>
</tr>
<tr>
<td>3-1</td>
<td>Various Stages of Coulomb Friction-Viscous Damping Model (from Reichman and Reinhorn 1994)</td>
<td>3-4</td>
</tr>
<tr>
<td>4-1</td>
<td>Perspective View of 1:3 scale R/C Frame Structure (a) Before Conventional Retrofit (b) After Conventional Retrofit</td>
<td>4-1</td>
</tr>
<tr>
<td>4-2</td>
<td>Building Dimensions and Location of Local Measuring Devices in Columns</td>
<td>4-4</td>
</tr>
</tbody>
</table>
### LIST OF ILLUSTRATIONS (cont'd)

<table>
<thead>
<tr>
<th>FIG.</th>
<th>TITLE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>4-3</td>
<td>Conventional Retrofit by Jacketing of Interior Columns (from Bracci 1992)</td>
<td>4-5</td>
</tr>
<tr>
<td>4-4</td>
<td>Detail of Conventional Retrofit with Concrete Jacketing and Joint Fillet (from Bracci 1992)</td>
<td>4-6</td>
</tr>
<tr>
<td>4-5</td>
<td>Perspective View of the Frame with Installed Damping Devices (a) with Tekton Friction Dampers; (b) with Sumitomo Friction Dampers</td>
<td>4-9</td>
</tr>
<tr>
<td>4-6</td>
<td>Location of Dampers and Measuring Devices</td>
<td>4-10</td>
</tr>
<tr>
<td>4-7</td>
<td>Different Configurations of the Tested Model</td>
<td>4-11</td>
</tr>
<tr>
<td>4-8</td>
<td>Perspective View of Tekton Friction Dampers Installed in the Mid-bay of the Frame</td>
<td>4-12</td>
</tr>
<tr>
<td>4-9</td>
<td>Installation Detail of a Tekton Friction Damper in the Mid-bay of the Frame</td>
<td>4-12</td>
</tr>
<tr>
<td>4-10</td>
<td>Perspective View of Sumitomo Friction Dampers Installed in the Mid-bay of the Frame</td>
<td>4-13</td>
</tr>
<tr>
<td>4-11</td>
<td>Installation Detail of a Sumitomo Friction Damper in the Mid-bay of the Frame</td>
<td>4-13</td>
</tr>
<tr>
<td>4-12</td>
<td>Simulated Ground Motion El-Centro S00E Scaled to PGA 0.3g</td>
<td>4-23</td>
</tr>
<tr>
<td>4-13</td>
<td>Elastic Response Spectra of Simulated El-Centro Earthquake (PGA=0.3g)</td>
<td>4-24</td>
</tr>
<tr>
<td>4-14</td>
<td>Simulated Ground Motion Taft N21E Earthquake PGA 0.2g</td>
<td>4-25</td>
</tr>
<tr>
<td>4-15</td>
<td>Elastic Response Spectra of Simulated Taft Earthquake (PGA 0.2g)</td>
<td>4-26</td>
</tr>
<tr>
<td>4-16</td>
<td>Simulated Ground Motion Mexico City Earthquake PGA 0.2g</td>
<td>4-27</td>
</tr>
<tr>
<td>4-17</td>
<td>Elastic Response Spectra of Simulated Mexico City Earthquake (PGA=0.2g)</td>
<td>4-28</td>
</tr>
<tr>
<td>4-18</td>
<td>Simulated Ground Motion Hachinohe N00S Scaled to PGA 0.3g</td>
<td>4-29</td>
</tr>
<tr>
<td>4-19</td>
<td>Elastic Response Spectra of Simulated Hachinohe Earthquake (PGA=0.3g)</td>
<td>4-30</td>
</tr>
<tr>
<td>4-20</td>
<td>Simulated Ground Motion Pacoima S16E Earthquake PGA 0.3g</td>
<td>4-31</td>
</tr>
<tr>
<td>4-21</td>
<td>Elastic Response Spectra of Simulated Pacoima Earthquake (PGA 0.3g)</td>
<td>4-32</td>
</tr>
<tr>
<td>4-22</td>
<td>Transfer Function from White Noise Ground Motion (with and w/o Tekton Friction Dampers)</td>
<td>4-37</td>
</tr>
<tr>
<td>4-23</td>
<td>Transfer Function from White Noise Ground Motion (with and w/o Sumitomo Friction Dampers)</td>
<td>4-39</td>
</tr>
<tr>
<td>4-24</td>
<td>Transfer Function from El-Centro PGA 0.3 Ground Motion (with and w/o Tekton Friction Dampers)</td>
<td>4-41</td>
</tr>
<tr>
<td>FIG.</td>
<td>TITLE</td>
<td>PAGE</td>
</tr>
<tr>
<td>------</td>
<td>----------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>4-25</td>
<td>Transfer Function from El-Centro PGA 0.3 Ground Motion (with and w/o</td>
<td>4-42</td>
</tr>
<tr>
<td></td>
<td>Sumitomo Friction Dampers)</td>
<td></td>
</tr>
<tr>
<td>4-26</td>
<td>Comparison of Displacement Response History for Structure without and</td>
<td>4-48</td>
</tr>
<tr>
<td></td>
<td>with Six Tekton Friction Dampers, from El-Centro Earthquake PGA 0.3g</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Test</td>
<td></td>
</tr>
<tr>
<td>4-27</td>
<td>Comparison of Acceleration Response History for Structure without and</td>
<td>4-49</td>
</tr>
<tr>
<td></td>
<td>with Six Tekton Friction Dampers, from El-Centro Earthquake PGA 0.3g</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Test</td>
<td></td>
</tr>
<tr>
<td>4-28</td>
<td>Comparison of Displacement Response History for Structure without and</td>
<td>4-50</td>
</tr>
<tr>
<td></td>
<td>with Six Sumitomo Friction Dampers, from El-Centro Earthquake PGA</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.3g Test</td>
<td></td>
</tr>
<tr>
<td>4-29</td>
<td>Comparison of Acceleration Response History for Structure without and</td>
<td>4-51</td>
</tr>
<tr>
<td></td>
<td>with Six Sumitomo Friction Dampers, from El-Centro Earthquake PGA</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.3g Test</td>
<td></td>
</tr>
<tr>
<td>4-30</td>
<td>Comparison of Displacement Response History for Structure without and</td>
<td>4-52</td>
</tr>
<tr>
<td></td>
<td>with Four Tekton Friction Dampers, from El-Centro Earthquake PGA</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.3g Test</td>
<td></td>
</tr>
<tr>
<td>4-31</td>
<td>Comparison of Acceleration Response History for Structure without and</td>
<td>4-53</td>
</tr>
<tr>
<td></td>
<td>with Four Tekton Friction Dampers, from El-Centro Earthquake PGA</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.3g Test</td>
<td></td>
</tr>
<tr>
<td>4-32</td>
<td>Comparison of Displacement Response History for Structure without and</td>
<td>4-54</td>
</tr>
<tr>
<td></td>
<td>with Two Tekton Friction Dampers, from El-Centro Earthquake PGA</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.3g Test</td>
<td></td>
</tr>
<tr>
<td>4-33</td>
<td>Comparison of Acceleration Response History for Structure without and</td>
<td>4-55</td>
</tr>
<tr>
<td></td>
<td>with Two Tekton Friction Dampers, from El-Centro Earthquake PGA</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.3g Test</td>
<td></td>
</tr>
<tr>
<td>4-34</td>
<td>Displacement Time History Response of the Model without and with Six</td>
<td>4-56</td>
</tr>
<tr>
<td></td>
<td>Tekton Friction Dampers, from Taft Earthquake PGA 0.2g Test</td>
<td></td>
</tr>
<tr>
<td>4-35</td>
<td>Acceleration Time History Response of the Model without and with Six</td>
<td>4-57</td>
</tr>
<tr>
<td></td>
<td>Tekton Friction Dampers, from Taft Earthquake PGA 0.2g Test</td>
<td></td>
</tr>
<tr>
<td>4-36</td>
<td>First Floor Single Damper Response, Taft Earthquake PGA 0.2g</td>
<td>4-58</td>
</tr>
<tr>
<td>4-37</td>
<td>Displacement Time History Response of the Model without and with Six</td>
<td>4-59</td>
</tr>
<tr>
<td></td>
<td>Tekton Friction Dampers, from Taft Earthquake PGA 0.4g Test</td>
<td></td>
</tr>
<tr>
<td>4-38</td>
<td>Acceleration Time History Response of the Model without and with Six</td>
<td>4-60</td>
</tr>
<tr>
<td></td>
<td>Tekton Friction Dampers, from Taft Earthquake PGA 0.4g Test</td>
<td></td>
</tr>
<tr>
<td>4-39</td>
<td>First Floor Single Damper Response, Taft Earthquake PGA 0.4g</td>
<td>4-61</td>
</tr>
<tr>
<td>FIG.</td>
<td>TITLE</td>
<td>PAGE</td>
</tr>
<tr>
<td>------</td>
<td>----------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>4-40</td>
<td>Displacement Time History Response of the Model without and with Six Tekton Friction Dampers, from Mexico City Earthquake PGA 0.1g Test</td>
<td>4-62</td>
</tr>
<tr>
<td>4-41</td>
<td>Acceleration Time History Response of the Model without and with Six Tekton Friction Dampers, from Mexico City Earthquake PGA 0.1g Test</td>
<td>4-63</td>
</tr>
<tr>
<td>4-42</td>
<td>First Floor Single Damper Response, Mexico City Earthquake PGA 0.1g</td>
<td>4-64</td>
</tr>
<tr>
<td>4-43</td>
<td>Displacement Time History Response of the Model without and with Six Tekton Friction Dampers, from Mexico City Earthquake PGA 0.2g Test</td>
<td>4-65</td>
</tr>
<tr>
<td>4-44</td>
<td>Acceleration Time History Response of the Model without and with Six Tekton Friction Dampers, from Mexico City Earthquake PGA 0.2g Test</td>
<td>4-66</td>
</tr>
<tr>
<td>4-45</td>
<td>First Floor Single Damper Response, Mexico City Earthquake PGA 0.2g</td>
<td>4-67</td>
</tr>
<tr>
<td>4-46</td>
<td>Displacement Time History Response of the Model without and with Six Tekton Friction Dampers, from Hachinohe Earthquake PGA 0.3g Test</td>
<td>4-68</td>
</tr>
<tr>
<td>4-47</td>
<td>Acceleration Time History Response of the Model without and with Six Tekton Friction Dampers, from Hachinohe Earthquake PGA 0.3g Test</td>
<td>4-69</td>
</tr>
<tr>
<td>4-48</td>
<td>First Floor Single Damper Response, Hachinohe Earthquake PGA 0.3g</td>
<td>4-70</td>
</tr>
<tr>
<td>4-49</td>
<td>Displacement Time History Response of the Model without and with Six Tekton Friction Dampers, from Pacoima Earthquake PGA 0.3g Test</td>
<td>4-71</td>
</tr>
<tr>
<td>4-50</td>
<td>Acceleration Time History Response of the Model without and with Six Tekton Friction Dampers, from Pacoima Earthquake PGA 0.3g Test</td>
<td>4-72</td>
</tr>
<tr>
<td>4-51</td>
<td>First Floor Single Damper Response, Pacoima Earthquake PGA 0.3g</td>
<td>4-73</td>
</tr>
<tr>
<td>4-52</td>
<td>Forces in Structural Components at First Floor, from El-Centro PGA 0.3g Test</td>
<td>4-74</td>
</tr>
<tr>
<td>4-53</td>
<td>Energy Distribution in Structure w/o and with Six Tekton Friction Dampers</td>
<td>4-77</td>
</tr>
<tr>
<td>4-54</td>
<td>Axial Force Fluctuation in First Floor Interior Column for Simulated Earthquake El-Centro 0.3g</td>
<td>4-78</td>
</tr>
<tr>
<td>4-55</td>
<td>Forces in Column vs Structural Capacity for El-Centro PGA 0.3g</td>
<td>4-79</td>
</tr>
<tr>
<td>5-1</td>
<td>An Extensive Hysteretic Model with Stiffness and Strength Deterioration and Pinching Due to Crack Opening and Closing</td>
<td>5-3</td>
</tr>
<tr>
<td>5-2</td>
<td>A Non-symmetric Distributed Plasticity Model Obtained through a Distributed Flexibility Model</td>
<td>5-4</td>
</tr>
<tr>
<td>5-3</td>
<td>Comparison of Experimental and Analytical Displacement for El-Centro 0.3g (with Six Tekton Friction Dampers)</td>
<td>5-14</td>
</tr>
<tr>
<td>FIG.</td>
<td>TITLE</td>
<td>PAGE</td>
</tr>
<tr>
<td>------</td>
<td>----------------------------------------------------------------------</td>
<td>-------</td>
</tr>
<tr>
<td>5-4</td>
<td>Comparison of Experimental and Analytical Acceleration for El-Centro 0.3g (with Six Tekton Friction Dampers)</td>
<td>5-15</td>
</tr>
<tr>
<td>5-5</td>
<td>Comparison of Experimental and Analytical Displacement for Taft 0.2g (with Six Tekton Friction Dampers)</td>
<td>5-16</td>
</tr>
<tr>
<td>5-6</td>
<td>Comparison of Experimental and Analytical Acceleration for Taft 0.2g (with Six Tekton Friction Dampers)</td>
<td>5-17</td>
</tr>
<tr>
<td>5-7</td>
<td>Comparison of Damper Forces for El-Centro Earthquake PGA 0.3g (with Six Tekton Friction Dampers)</td>
<td>5-18</td>
</tr>
<tr>
<td>5-8</td>
<td>Structural Resistance in Presence of Friction Dampers</td>
<td>5-19</td>
</tr>
<tr>
<td>6-1</td>
<td>Composite Response Spectra for SDOF</td>
<td>6-3</td>
</tr>
<tr>
<td>6-2</td>
<td>Composite Response Spectra for MDOF</td>
<td>6-7</td>
</tr>
<tr>
<td>6-3</td>
<td>Response-Demand Using Composite Spectra</td>
<td>6-9</td>
</tr>
<tr>
<td>6-4</td>
<td>Demand in Inelastic Structure Using Composite Spectra</td>
<td>6-12</td>
</tr>
<tr>
<td>6-5</td>
<td>Composite Spectra vs Capacity of Structure for Taft 0.05g, 0.20g and 0.30g for 2% and 10% Critical Damping. Tested Damping Ratios 4.6%, 8.2% and 3% for above Motions, Respectively.</td>
<td>6-13</td>
</tr>
<tr>
<td>6-6</td>
<td>Cyclic Hysteretic Energy Dissipation</td>
<td>6-15</td>
</tr>
<tr>
<td>6-7</td>
<td>Influence of Supplemental Damping</td>
<td>6-19</td>
</tr>
<tr>
<td>6-8</td>
<td>Evaluation of Structural Response for El-Centro Earthquake, PGA 0.3g</td>
<td>6-21</td>
</tr>
<tr>
<td>6-9</td>
<td>Evaluation of Structural Response for Taft Earthquake, PGA 0.2g</td>
<td>6-22</td>
</tr>
<tr>
<td>6-10</td>
<td>Evaluation of Response Using NEHRP Spectra (PGA=0.3g)</td>
<td>6-24</td>
</tr>
<tr>
<td>6-11</td>
<td>Evaluation of Response Using NEHRP Spectra (PGA=0.2g)</td>
<td>6-25</td>
</tr>
<tr>
<td>6-12</td>
<td>Summary of Experimental Response of Tested Structure Model (El-Centro, PGA 0.3g)</td>
<td>6-26</td>
</tr>
<tr>
<td>6-13</td>
<td>Summary of Experimental Response of Tested Structure Model (Taft, PGA 0.2g)</td>
<td>6-27</td>
</tr>
<tr>
<td>6-14</td>
<td>Summary of Experimental Response of Tested Structure Model with Various Dampers (El-Centro, PGA 0.3g)</td>
<td>6-30</td>
</tr>
<tr>
<td>6-15</td>
<td>Summary of Experimental Response of Tested Structure Model with Various Dampers (Taft, PGA 0.2g)</td>
<td>6-31</td>
</tr>
<tr>
<td>A-1</td>
<td>Layout of Slab Steel Reinforcement</td>
<td>A-2</td>
</tr>
<tr>
<td>A-2a</td>
<td>Details of the Beam Steel Reinforcement</td>
<td>A-3</td>
</tr>
<tr>
<td>A-2b</td>
<td>Details of the Beam Steel Reinforcement (Continued)</td>
<td>A-4</td>
</tr>
</tbody>
</table>
## LIST OF ILLUSTRATIONS (cont'd)

<table>
<thead>
<tr>
<th>FIG.</th>
<th>TITLE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-3</td>
<td>Details of the Column Steel Reinforcement</td>
<td>A-5</td>
</tr>
<tr>
<td>A-4</td>
<td>Gradation Analysis of the Concrete Mix</td>
<td>A-6</td>
</tr>
<tr>
<td>A-5</td>
<td>Average Concrete Specimen Strength Versus Time</td>
<td>A-6</td>
</tr>
<tr>
<td>A-6</td>
<td>Measured Representative Stress-Strain Relationships of the Reinforcing Steel</td>
<td>A-8</td>
</tr>
<tr>
<td>B-1</td>
<td>Instrumentation and Locations</td>
<td>B-3</td>
</tr>
</tbody>
</table>
SECTION 1

INTRODUCTION

Many reinforced concrete frame structures, designed according to old standards have deficient nonductile details that make them vulnerable to future seismic events. Based on conventional seismic design practice, a structure is capable to survive a severe earthquake without collapse at the expense of allowing inelastic action in specially detailed critical regions of gravity load carrying members such as columns and beams near or adjacent to the beam-column joints. Inelastic behavior in these regions, though able to dissipate substantial energy, often results in significant damage to the structural members. The inter-story drifts required to achieve significant hysteretic energy dissipation in critical regions are large and usually result in permanent deformations and substantial damage to non-structural elements such as infill walls, partitions, doorways, and ceilings.

An innovative approach for earthquake hazard mitigation was introduced by adding protective devices, non-load bearing, to redistribute the energy within the structure. During a seismic event, the finite energy input is transformed partially into kinetic (movement) and potential (stored) energy and partially dissipated through structure is inherent damping (heat) and through hysteresis in gravity load carrying elements experiencing inelastic deformations. This last energy component, i.e. the hysteretic, is responsible for reducing the structure capacity of carrying gravity loads and its lateral strength or deformation capacities, thus increasing the demand-capacity ratios near collapse. The structural performance can be improved if the total energy input is reduced,
or a substantial amount can be dissipated by supplemental damping devices (non-gravity load bearing), and not by the gravity load bearing structural members.

The energy balance equation (Uang and Bertero 1990) can be readjusted to include the effect of damping devices:

\[ E_I = E_K + E_s + E_D + E_H + E_{SD} \]  

(1-1)

where \( E_I = \int (m \ddot{u}_s) \ddot{u}_s \) is the total input energy, \( E_K = \frac{m (\ddot{u}_t)^2}{2} \) is the 'absolute' kinetic energy, \( E_s = \frac{(f_s)^2}{2k} \) is the elastic strain energy in the structure, \( E_D = \int c \dot{u}^2 dt \) is energy dissipated through structural damping, \( E_H \) is the total hysteretic energy dissipated in the structure and \( E_{SD} \) is the energy dissipated by supplemental damping devices.

The total absolute energy input, \( E_I \), is the work done by the base shear over the foundation ground movement. This energy contains the inertial forces in the structure, including the response amplifications.

In absence of supplemental damping, the inelastic response and the hysteretic energy demand increase. However, besides the negative effect of increased damage in the structural members, associated with the hysteretic energy dissipation, this increase has a positive effect in softening the structure, thus reducing the inertia forces and the total energy input. This effect is on the base of current seismic design provision which allow for inelastic response. Both energy input reduction and reduction of hysteretic energy demand (thus reducing damage) can be obtained through modern protective devices. The
recently developed seismic base isolation (Buckle 1990, Kelly 1991, Mokha et al. 1991) accomplishes the task of reducing the total energy input by filtering the input motion into the structure at its base and by dissipating part of this energy at same location through local damping. The reduction of the energy input reduces the demand for energy dissipation through inelastic action and hysteretic excursions. In most cases inelastic action in the superstructure is avoided completely.

More recently developed devices accomplish redistribution of internal energy, or reduce substantially the total energy input, through active means, such as dampers or active braces (Soong 1990, Reinhorn 1992). These devices, incorporated in complex control systems, act based on "real time" processed information from sensors, which anticipate the further structural movements. Although such systems are extremely efficient in small structures they require additional energy, sometimes unreliable or expensive, in order to produce the energy redistribution in large structures.

Another approach to improve performance and damage control through altering the energy distribution are supplemental damping devices. These mechanical devices are incorporated in the frame of structure and dissipate energy throughout its height. These devices dissipate energy by either yield of mild steel, sliding friction, viscoelastic action in polymeric materials, piston or plate movement within fluid, or fluid transfer through orifices. These systems are the subject of the current research.
1.1 Viscoelastic devices

Viscoelastic dampers, made of bonded viscoelastic layers (acrylic polymers) have been developed by 3M Company Inc. and were used in wind and seismic applications. Examples are the World Trade Center in New York City (110 stories), Columbia SeeFirst Building in Seattle (73 stories), the Number Two Union Square Building in Seattle (60 stories), and General Service Administration Building in San Jose (13 stories). A two-story new school building in Phoenix, Arizona has been constructed in 1992 with its beam to column connections incorporating viscoelastic materials as shown in Fig. 1-1.

The characteristics and suitability of viscoelastic dampers to enhance performance of structures were studied by Lin et al. 1988, Aiken et al. 1990, Chang et al. 1991 and Lobo et al. 1993. Fig. 1-2 shows a typical damper and an installation detail in a steel structure.

The behavior of viscoelastic dampers is controlled by the shear of viscoelastic layers. The acrylic material exhibits solid viscoelastic behavior with storage and loss (stiffness) moduli dependent on frequency and temperature.

In the aforementioned studies, 3M Company's dampers were used. Other devices developed by Lorant Group were studied by Hsu, 1992. Hazama Corp. in Japan developed similar devices using similar materials (Fujita 1991). Shimizu Corporation
Figure 1-1  Detail of Beam to Column Connection with Viscoelstic Material
Figure 1-2  Viscoelastic Damper and Installation Detail
developed viscoelastic walls, in which solid thermoplastic rubber sheets are sandwiched between steel plates (Fujita 1991).

The use of dampers in elastic structures was proven efficient, in particular when the inherent damping of the structure is low (Aiken 1990). The use of dampers in inelastic structures, studied by Lobo et al. (1993), Wood et al. (1994) indicate that the viscoelastic material dissipates large amount of energy reducing the demand for hysteretic energy dissipation. In gravity load carrying components, the damping index (equivalent to critical damping in elastic structures) reaches 20% to 22%. However, the overall base shear in the structure has the tendency to increase or only minimally decrease in presence of dampers.

1.2 Viscous walls

Viscous damping walls, consisting of a plate floating in a thin case made of steel plates (the wall) filled with highly viscous fluid (see Fig. 1-3), have been developed by Sumitomo Construction Company, Ltd., and the Buildings Research Institute in Japan. The walls were investigated by Sumitomo Construction Company (Arima, 1988) and were already used in a 78.6 m high, 14 story building at the center of Shizuoka city, 150 km west of Tokyo, Japan. Earthquake simulator tests of a 5 story, reduced-scale building model and a 4-story, full-scale steel frame building embedding such walls have been carried out (Arima, 1988) and the most recently, a three story 1:3 scale reinforced concrete structure has been tested in Seismic Simulation Laboratory at the State
Figure 1-3  Viscous Wall, Installation Detail and Hysteresis Loops (from Miyazaki, 1992)
University of New York at Buffalo (Reinhorn et al. 1994). The devices exhibit nonlinear viscous behavior with stiffening characteristics at high frequencies.

1.3 Fluid Viscous Dampers

Fluid viscous dampers have been used in military for many years because of their efficiency and longevity. This kind of devices operates on the principle of fluid flow through orifices. The construction of this kind of dampers is shown in Fig. 1-4 and the typical force-displacement relationship from test is shown in Fig. 1-5.

The first production usage of a hydraulic damper was in the 75 mm French artillery rifle of 1897. The damper was used to reduce recoil forces and had a stroke of over 18 inches. The modern fluid damping devices are just beginning to emerge in large scale structural construction in recent years. The device possesses linear or nonlinear viscous behavior upon design and is relatively insensitive to temperature changes. The force in a fluid damper may be expressed as:

\[ P = C|\dot{u}|^6 \text{sgn}(\dot{u}) \]

The size of the device is very compact in comparison to force capacity and stroke. Experimental and analytical studies of buildings and bridge structures incorporating the damping devices fabricated by Taylor Devices, Inc., have recently been performed (Constantinou et al. 1993, Reinhorn et al. 1995). Very large reductions of elastic response
Fluid Viscous Damper Construction and Installation Detail

Figure 1-4    Fluid Viscous Damper Construction and Installation Detail
Figure 1.5 Typical Hysteresis Loops of Fluid Viscous Damper

- Temperature $= 23^\circ C$
- $f = 4 \text{ Hz}$
- $f = 2 \text{ Hz}$
- $f = 1 \text{ Hz}$

Displacement (in) vs. Force (lb)
were achieved by the introduction of these devices. The feature of a pure viscous damper that the damping force is out-of-phase with the displacement can be a particularly desirable attribute for passive damping applications to buildings. The Travelers Hotel, a landmark hotel built in the 1920’s in Sacramento, California, has been designed with fluid viscous dampers as part of its seismic retrofit scheme. Construction has not yet started.

Nonlinear viscous behavior can be achieved through specially shaped orifice to alter the flow characteristics with fluid speed. Fluid dampers with nonlinear characteristics have been adopted in a number of projects in U.S. recently. The San Bernardino County Medical Center in California is a five building isolated complex utilizing 400 high damping rubber bearings and 233 nonlinear viscous dampers with $\alpha=0.5$. Furthermore, studies for the seismic retrofit of the suspended part of the Golden Gate bridge in San Francisco concluded that the use of fluid dampers with $\alpha=0.75$ produce the desired performance (Rodriquez 1994).

1.4 Hysteretic Devices

Hysteretic devices are devices which can dissipate energy through inelastic deformations of their components or friction within their parts or properly designed surfaces.
1.4.1 Friction Devices

There are a variety of friction devices which have been proposed for structural energy dissipation. Usually friction systems generate rectangular hysteresis loops characteristics of Coulomb friction. Typically these devices have very good performance characteristics, and their behavior is not significantly affected by load amplitude, frequency, or the number of applied load cycles. The devices differ in their mechanical complexity and the materials used for the sliding surfaces.

A friction device located at the intersection of cross bracing has been proposed by Pall (1982, 1987) and used in six building in Canada. Fig. 1-6 illustrates the design of this device. When seismic load is applied the interior deforms into a parallelogram and friction is produced at the bolts location. Experimental studies by Filiatrault (1985) and Aiken (1988) confirmed that these friction devices could enhance the seismic performance of structures. The devices provided a substantial increase in energy dissipation capacity and reduced drifts in comparison to moment resisting frames. Reduction in story shear forces were moderate in inelastic structures. However, these forces are primarily resisted by the braces in a controlled manner and only indirectly resisted by the primary structural elements.

Friction devices have been developed and manufactured for many years by Sumitomo Metal Ltd., Japan. The original application of these devices was in railway rolling stock bogie trucks. It is only since the mid of 1980's that the friction dampers have
Figure 1-6  Pall Friction Damper, Typical Hysteretic Loop and Application
been extended to the field of structural and seismic engineering. A detailed description of this kind of devices is presented in Section 2.

Recently, a similar type of friction dampers, manufactured by Tekton company, Arizona, was tested in the Seismic Simulation Laboratory at the State University of New York at Buffalo. This type of dampers is made of simple components similar to those by Constantinou and Reinhorn et al. (1991) designed to minimize the cost of manufacturing. The "yielding" force of the damper, i.e. the friction level, can be adjusted through the appropriate torque of bolts that control the pressure on the friction surfaces. A detailed description of this kind of devices is presented in Section 2.

Friction dampers were suggested as displacement control devices for bridge structure with sliding supports (Constantinou, Reinhorn, et al. 1991a, 1991b) made of steel-bronze surface (see Fig. 1-7). The devices can be adjusted to provide a desirable level of resistance and stable energy dissipation in numerous cycles.

Another friction device, proposed by Fitzgerald (1989), utilize slotted bolted connections (SBCs) in concentrically braced connections. Slotted Bolted Connections are modified bolted connections designed to dissipate energy through friction in rectilinear tension and compression loading cycles. Components tests demonstrated stable friction behavior. It may be noted that the sliding interface is that of steel on steel. Very recently Grigorian (1993) tested a slotted bolted connection (see Fig. 1-8) which was nearly identical to the one Fitzgerald (1989) expect for the sliding interface which consisted of
Figure 1-7 Detail of Displacement Controller
HARDENED WASHER
B-EH-112 SOLON
COMPRESSIO N WASHERS
UNDER NUT

1/8" TH. BRASS INSERT PLATES:

DIRECT TENSION INDICATOR WASHER (DTI), UNDER HEAD

ALL PLATES ARE OF 5/8" TH. A36 STEEL

1/2" DIA. A325 BOLT, 3-1/2" LONG

WELL:

9/16"x3-1/2" LONG SLOT

STEEL ON STEEL HYSTERESIS DIAGRAM

STEEL ON BRASS HYSTERESIS DIAGRAM

Figure 1-8   Detail of a Slotted Bolted Connection and Hysteresis Behavior
brass in contact with steel. This interface exhibits more stable frictional characteristics than the steel on steel interface.

A more complex friction device (Energy Dissipation Restrain, EDR) combined with self centering capabilities provided by internal springs and end gaps (see Fig. 1-9) was developed by Flour Daniel Corp. (Nims, 1993). This device can develop X type hysteretic loops with restoring capabilities.

All the friction devices described above utilize sliding interfaces consisting of steel-on-steel, brass on steel, graphite impregnated bronze on stainless steel. The composition of the interface is of extraordinary importance for insuring longevity of operation of these devices. Low carbon alloy steels (common steels) will corrode and the interface properties will change with time. Moreover, brass or bronze promote additional corrosion when it is in contact with low carbon steels (BSI, 1979). Only authentic stainless steels with high chromium content do not suffer additional corrosion in contact with brass or steel and could be used for long term operation. At the same time Teflon PTFE and steel interface are inert to reciprocal corrosion and have long term stable properties. Moreover, those interfaces have lower friction coefficient and require larger pressure on the interfaces (Tsopelas 1994).

1.4.2 Elastomeric Spring Dampers

A type of single-acting damper device used previously in the railroad and steel industries is studied recently by Pekcan et al. (1995) in the Seismic Simulation Laboratory at the State University of New York at Buffalo. These devices dampers which contain a
Figure 1.9  Energy Dissipating Restraint and Representative Force-Displacement Loops (from Nims 1993)
silicone-based elastomer, called elastomeric spring, were modified to operate in a double-acting fashion in the study. The dampers can be designed to give both spring and hysteretic behavior. Fig. 1-10 shows the physical arrangement of the double-acting damper and the typical hysteric loops of the damper.

1.4.3 Metallic Systems

This category of energy dissipation systems takes advantage of the hysteretic behavior of mild steel when deform into their post-elastic range. A wide variety of different types of devices utilizing flexural, shear or extensional deformation mode into the inelastic range. A particularly desirable feature of these system is their stable behavior, long term reliability, and generally good resistance to environmental and temperature factors.

1.4.3.1 Yielding Steel Elements

The ability of mild steel to sustain many cycles of stable yielding behavior has led to the development of a wide variety of devices which utilize this behavior to dissipate seismic energy (Kelly et al. 1972, Skinner et al. 1980, Henry 1978, 1986, Tyler 1983, 1985). Many of these devices use mild steel plates with triangular or hourglass shapes (Tyler 1978, Stiemer et al. 1981) so that the yielding is spread almost uniformly throughout the material. This results in a device which is able to sustain repeated inelastic deformations in a stable manner, avoiding concentrations of yielding and premature failure and buckling of braces, hence, pinched hysteretic behavior does not occur. An energy absorbing device in the form of round mild steel rod with a rectangular shape (Fig.1-11)
Figure 1-10  Elastomeric Spring Damper and Hysteresis Behavior
Figure 1-11  Detail of a Yielding Steel Bracing System in a Building in New Zealand
introduced at the intersection of cross bracing, have been developed in New Zealand (Tyler 1978, Skinner 1980). Some of these devices were tested on shaking table at U.C. Berkeley as parts of seismic systems (Kelly 1980). They have been incorporated in a number of buildings in New Zealand and similar ones were widely used in seismic isolation applications in Japan (Kelly 1988).

One such device that uses X-shaped steel plates is the Bechtel Added Damping and Stiffness (ADAS) devices. ADAS elements are an evolution of an earlier use of X-plates, as damping supports for piping systems (Stiemer, et al., 1981). Extensive experimental studies have investigated the behavior of individual ADAS elements and structural systems incorporating ADAS elements (Bergman and Goel, 1987, Whittaker, et al., 1991). The tests showed stable hysteretic performance (Fig. 1-12). ADAS devices had been installed in two bay-story, non-ductile reinforced-concrete building in San Francisco as a part of a seismic retrofit (Fiero et al. 1993), and in two building in Mexico City. The principal characteristics which affect the behavior of an ADAS devices are its elastic stiffness, yielding strength, and yield displacement. ADAS devices are usually mounted as part of a bracing system, which must be substantially stiffer than the surrounding structure elements. The introduction of such a heavy bracing system into a structure may be prohibitive.

Triangular-plate energy dissipaters were originally developed and used as the damping elements in several base isolation applications (Boardman et al. 1983). The triangular plate concept has been extended to building dampers in the form of triangular ADAS, or T-ADAS, element (Tsai and Hong 1992). Component tests of T-ADAS
elements and pseudo-dynamic tests of a two-story frame have shown very good results (Fig. 1-13). The T-ADAS device embodies a number of desirable features: no rotational restraint is required at the top of the brace connection assemblage, and there is no potential for instability of the triangular plate due to excessive axial load in the devices.

An energy dissipater for cross-braced structures, which uses mild steel round bars or flat plates as the energy absorbing element, has been developed by (Tyler 1985). This concept has been applied to several industrial warehouses in New Zealand. A number of variations on the steel cross-bracing dissipater concept have been developed in Italy (Ciampi 1991). A 29-story steel suspension building (with floors "hung" from the central tower) in Naples, Italy, utilizes tapered steel devices as dissipaters between the core and the suspended floors.

A six-story government building in Wanganui, New Zealand, used steel-tube energy-absorbing devices in precased concrete cross-braced panels (Matthewson and Davey 1979). The devices were designed to yield axially at a given force level. Recent studies have experimentally and analytically investigated a number of different cladding connection concepts (Craig et al. 1992).

Several types of mild steel energy dissipaters have been developed in Japan (Kajima Corp. 1991, Kobori et al. 1988). So-called honeycomb dampers have been incorporated in 15-story and 29-story buildings in Tokyo. Honeycomb dampers are X-plates (either single plates, or multiple plates connected side by side) that are loaded in plane of the X. (This is orthogonal to the loading direction for triangular or ADAS X-
plates). Kajima Corporation has also developed two types of omni-directional steel dampers, called "Bell" dampers and "Tsudumi" dampers (Kobori et al. 1988). The Bell damper is a single-tapered steel tube, and the Tsudumi damper is a double-tapered tube intended to deform in the same manner as an ADAS X-plate but in multiple directions. Bell dampers have been used in a massive 1600-ft long ski-slope structure to permit differential movement between four dissimilar parts of the structure under seismic loading while dissipating energy. Both of these applications are located in the Tokyo area.

Another type of joint damper for application between two buildings has been developed (Sakurai et al., 1992). The device is a short lead tube that is loaded to deform in shear (Fig. 1-14). Experimental investigations and an analytical study have been undertaken.

Particular issues of importance with metallic devices are the appropriate post-yield deformation range, such that a sufficient number of cycles of deformation can be sustained without premature fatigue, and the stability of the hysteretic behavior under repeated post-elastic deformation.

1.4.3.2 Lead Extrusion Devices (LEDs)

The extrusion of lead was identified as an effective mechanism for energy dissipation in the 1970s (Robinson and Greenbank 1976). LED hysteretic behavior is very similar to that of many friction devices, being essentially rectangular (Fig. 1-9). LEDs have been applied to a number of structures, for increasing the damping in seismic isolation system, and as energy dissipaters within multi-story buildings, In Wellington,
Figure 1-12  ADAS Devices Hysteresis Loops 
(from Whittaker, 1991)

Figure 1-13  T-ADAS Device Hysteresis Loops 
(from Tsai, 1992)

Figure 1-14  Lead Joint Damper and Hysteresis Loops 
(Sakurai, 1992)
New Zealand, a 10-story, cross-braced, concrete police station is base isolated, with sleeved-pile flexible elements and LED damping elements (Charleson et al. 1987). Several seismically-isolated bridges in New Zealand also utilize LEDs (Skinner et al. 1980). In Japan, LEDs have been incorporated in 17-story and 8-story steel frame buildings (Oiles Corp., 1991). The devices are connected between precased concrete wall panels and the surrounding structural frame.

LEDs have a number of particularly desirable features: their load-deformation relationship is stable and repeatable, being largely unaffected by the number of loading cycles; they are insensitive to environmental factors; and tests have demonstrated insignificant aging effects (Robinson and Cousins 1987) (Fig. 1-15).

1.4.3.3 Shape Memory Alloys (SMAs)

Shape memory alloys have the ability to "yield" repeatedly without sustaining any permanent deformation. This is because the material undergoes a reversible phase transformation as it deforms rather than intergranular dislocation, which is typical of steel. Thus, the applied load induce a crystal phase transformation, which is reversed when the load is removed (Fig. 1-16). This provides the potential for the development of simple devices which are self-centering and which perform repeatably for a large number of cycles.

Several earthquake simulator studies of structures with SMA energy dissipaters have been carried out. At the Earthquake Engineering Research Center of the University of California (Aiken et al. 1992), a 3-story steel model was tested with Nitinol (nickel-
titanium) tension devices as part of a cross-bracing system, and at the National Center for Earthquake Engineering Research (Witting and Cozzarelli 1992), a 5-story steel model was tested with copper-zinc-aluminum modes were investigated. Typical hysteresis loops from these tests are shown in Fig. 1-17. Results showed that the SMA dissipaters were effective in reducing the seismic responses of the models.

1.4.3.4 Eccentrically Braced Frame (EBF)

Steel moment-resisting frames have been regarded by structural designers for their earthquake-resistant behavior. However moment-resisting frames tend to be flexible, braced frames are considered as a mean of providing increased structural stiffness. Although Concentrically Braced Frames (CBFs) can easily provide the needed stiffness, the cyclic inelastic behavior of concentrically braced frames is strongly influenced by the cyclic post-buckling behavior of individual braces (Popov et al. 1976). Eccentrically Braced Frames (EBFs) have emerged as a well recognized and widely used structural system for resisting lateral seismic forces. Hysteretic behavior is concentrated in specially designed regions (shear links) of EBF (see Fig. 1-18) and other structural elements are designed according to capacity design principle and intended to remain elastic under all but the most severe excitations. Extensive research has been devoted to EBF (Roeder et al. 1978, Popov et al. 1987, Whittaker et al. 1987) and the concept has seen rapid recognition and acceptance by the structural engineering profession since the inclusion of design rules into seismic code of practice. These braces are using, however, some parts of
Figure 1-15  LED Hysteresis Loops
(from Robinson, 1987)

Figure 1-16  SMA Superelastic Hysteresis Behavior
(from Aiken, 1992)

Figure 1-17  NiTi (Tension) and Cu-Zn-Al (Torsion) Hysteresis Loops
(from Aiken, 1992, Witting, 1992)

1-29
Figure 1-18 Different Kind of Eccentrically Braced Element
the gravity load resisting elements which might need to be sacrificed in severe earthquake with implication of substantial damage.

1.5 Code Provision for Design of Structures Incorporating Passive Energy Dissipating Devices

It is imperative for implementation of the technology of energy dissipating devices to have a code design specification. Currently, such code specifications for structures with damping braces do not exist. The absence of such code specification may prevent widespread use of the technology. The existing codes, such as UBC and SEAOC have included provision for design of base isolation systems. Many codes, such as NEHRP, UBC and SEAOC, have included design of EBFs in their provisions. Efforts are made by code agencies (FEMA, ATC, SEAOC) to develop guidelines for use of dampers based on studies of elastic structures.

1.6 Objectives of This Investigation

The research was developed to:

1. investigate experimentally the behavior of friction dampers and structural response when the structural system experiences inelastic deformations.

2. model analytically the friction dampers as part of an inelastic structural model.

3. validate the analytical modeling using experimental data.
4. develop a simplified procedure to estimate the structural seismic demands in presence of dampers

5. determine the contribution of dampers to the changing of the demand-capacity relation (performance index) in severe ground shaking.
SECTION 2

FRICTION DAMPERS

2.1 Description of Tekton Friction Damping Devices

Friction dampers operate as steel shaft sliding between specially design friction pads. The cross sections of a typical Tekton friction damper are shown in Fig. 2-1. The dampers consist of series of adjustable bolts through which one can control the normal force applied to the friction pads. The yield force (break-through friction force) has linear relationship with applied torque on the adjustable bolts. The length of the dampers can be adjusted by pulling or pushing the steel shaft which gives the flexibility for installation.

2.2 Description of Sumitomo Friction Damping Devices

The longitudinal and cross-sections of a typical Sumitomo friction damper are shown in Fig. 2-2. The dampers consist of a series of wedges which act against each other under the load from a compressed spring and apply a normal force to the friction pads. The friction pads slide directly on the inner surface of the steel casing of the device. The friction pads are copper alloy with graphite plug inserts which provide dry lubrication to the unit, ensuring a stable friction force and reducing noise during movement.

For practical applications the devices are incorporated in structural braces or in structural joints with large deformations (see experimental study in Section 4).
Figure 2-1  Construction of Tekton Friction Damper
Figure 2-2
Sectional Views of a Sumitomo Friction Damper

(a) Longitudinal Section
(b) Cross Section

- inner wedge
- outer wedge
- cup spring
- outer cylinder
- outer wedge (in 3 parts)
- friction pad
2.3 Operation of Dampers

The force in a damper is a result of friction between sliding shaft and friction pads for Tekton friction dampers or between friction pads and the inner surface of the steel casing for Sumitomo friction device. The force in a damper can be defined as two stages, stick and slip, as:

\[ F_D = k_D U, \quad \text{for } |F_D| \leq \mu_{\text{break-away}} N \]  

where \( F_D \) is damper force, \( k_D \) is the stiffness of the damper, \( U \) is the deformation of the dampers, \( N \) is the normal force between steel shaft and friction pads (between friction pads and the inner surface of the steel casing for Sumitomo friction device), \( \mu_{\text{break-away}} \) is the maximum static friction coefficient and \( \mu_{\text{min}} \) is the friction coefficient at sliding stage.

\[ F_D = \mu_{\text{min}} N, \quad \text{for } \mu_{\text{min}} N < |F_D| < \mu_{\text{break-away}} N, \text{ after sliding occurred.} \]  

2.4 Testing of Damping Devices

Six Tekton friction devices were tested under frequencies 1 Hz, 2 Hz, 3 Hz and 4 Hz, and six Sumitomo friction devices were tested under frequencies 1 Hz, 1.5 Hz and 2 Hz. Each test consist of 20 cycles of harmonic (sinusoidal) motion. The devices were constructed for the retrofit of a reinforced concrete structural model. These devices were tested on a MTS testing machine using a series of harmonic displacements and the resisting forces were measured simultaneously. The purpose of the testing was to confirm the correct setting of slip force and to identify any dependency of the force-displacement
behavior on the loading frequency, temperature and number of loading cycles. The testing results are shown in Fig. 2-3, 2-4 and Table 2-1. The testing results show that the properties of the dampers are nearly independent on frequency, temperature and number of cycles.
Figure 2-3  Test Results of Tekton Friction Damper for Various Frequencies
Figure 2-4 Test Results of Sumitomo Friction Damper for Various Frequencies
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<th>Damper #</th>
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<th>Max. Force (kips)</th>
<th>Max. displ. (in)</th>
<th># of cycles</th>
<th>Location in model</th>
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2-8
SECTION 3

ANALYTICAL MODELING OF FRICTION DAMPERS

3.1 Mathematical Modeling

The shape of the force versus displacement loop of friction dampers is mostly rectangular with smooth corners as shown in the experimental results of Section 2.

3.1.1 Bouc-Wen’s Model

A friction damper force $F_D(t)$ can be represented as:

$$F_D = k_0(\alpha U + (1 - \alpha)ZU_y)$$

where $k_0$ is the initial stiffness, $\alpha$ is the ratio of post-yielding stiffness to initial pre-yielding stiffness, $U$ is the relative deformation in the damper, $U_y$ is the yield displacement of the damper and $Z$ is a nondimensional quantity given by:

$$Z = \left( \frac{\dot{U}}{U_y} \right) \left[ A - Z^n \left( \gamma \text{sgn}(\dot{U}Z) + \beta \right) \right]$$

in which $\eta$ is a parameter controls the transition shape from elastic range to yielding range. The value of this parameter can be increased to achieve near-bilinear behavior rather than smooth bilinear behavior ($\eta=2$ in this study). When $A/(\beta+\gamma)=1$ the model reduces to model of viscoplasticity (Constaintinou et al. 1990b, Ozedemir and Kelly 1976). The damper force $F_D$ can be calculated using semi-implicit Runge-Kutta method (Rosenbrook
1964) as in Section 5.2.2.1 in detail. To increase the computation speed, Reichman and Reinhorn, 1994 solved a close form solution for Eq. (3-1) and (3-2) for $\eta=2$. The nondimensional parameter $Z$ is obtained as:

\[ Z = \frac{a}{b} f \left( ab \left( \frac{U}{U_y} \right) + c \right) \]  

(3-3)

where,  

\[ a = \sqrt{A} ; \quad b = \sqrt{q} + \beta \text{sgn}(\dot{U}Z) ; \quad c = \text{constant of integration, equal to displacement at a previous branch, while } f[] \text{ is a function described as follows:} \]

\[ f[ ] = \begin{cases} 
\tanh[ ] & \text{for } \text{sgn}(\dot{U}Z) > 0; \quad b^2 Z^2 < a^2 \\
\coth[ ] & \text{for } \text{sgn}(\dot{U}Z) > 0; \quad b^2 Z^2 > a^2 \\
\tan[ ] & \text{for } \text{sgn}(\dot{U}Z) < 0 
\end{cases} \]  

(3-4)

The function $f[]$ controls the branch of the hysteretic loop, when $\text{sgn}(\dot{U}Z) > 0$, loading, occurs and when $\text{sgn}(\dot{U}Z) < 0$ unloading. The integration constant $c$ keeps record of the last transition point from one branch to the other.

3.1.2 Coulomb Friction-Viscous Damping Model (Reichman and Reinhorn 1994)

The friction damper force can be described in three stages and can be represented as a combination of three components: a linear rise, coulomb component and viscous damping component:

\[ F_D = k_J U + c_J \ddot{U} + \mu_J N \]  

(3-5)
where $F_D$ is the damper force, $N$ is the normal force, $k_j$, $c_j$ and $\mu_j$ are the stiffness, equivalent damping and friction coefficient at various stages of computation (see Fig. 3-1) as follows:

(a) Stick stage

For $j = 1$ (i.e. when $|F| \leq \mu_{\text{break-away}} N$):

$k_1 = k_0$, $c_1 = 0$ and $\mu_1 = 0$

(b) Transition stage

For $j = 2$ (i.e. $\mu_{\text{min}} N \leq |F| \leq \mu_{\text{max}} N$), after sliding occurred, then

$k_2 = 0$, $\mu_2 = \mu_{\text{min}}$, $c_2 = c_{eq}$

$c_{eq} = \frac{N(\mu_{\text{max}} - \mu_{\text{min}})}{\dot{U}_{\text{lim}it}}$

where $\dot{U}_{\text{lim}it}$ is a constant depending on surface properties and it can be obtained by inverting the exponent “a” in the model by Mokha et al. (1989). This “velocity” is usually between 2-4 in/sec. It should be noted that $N$ is variable with time and therefore $c_{eq}$ is also a time dependent variable.

(c) Sliding stage
Figure 3-1  Various Stages of Coulomb Friction-Viscous Damping Model (from Reichman and Reinhorn 1994)
The above stage applies when sliding occurs at velocities greater than the limit, $|\dot{U}| \geq \dot{U}_{\text{lim}}$. For velocities larger than $\dot{U}_{\text{lim}}$, then stage three, ($j=3$), applies:

$$k_3 = 0, \quad c_3 = 0, \quad \mu_3 = \mu_{\text{max}}.$$  

If the friction force drops below the $\mu_{\text{min}}$ N the system is transferred back to the first stage ($j=1$).

This model is able to represent in addition to the stick-slip condition, the smooth transition observed between slip and stick stages. The smooth transition in reality and in the analytical procedure, reduces instability and the influence of very high modes resulting from abrupt transition from slip to stick. It is noted that by varying the normal force, $N$, in the damper, the device can develop variable reaction. Such device can be used as part of a motion control scheme as a semi-active device.

### 3.1.3 Modeling of Tested Dampers

The dampers tested in this experimental study are “frequency independent. The dampers are calibrated to provide approximately 3.2 kips friction force, as shown in Table 2.1. The dampers are modeled using Bouc-Wen’s model in further analytical studies and compared with the Coulomb friction-viscous damping model. It should be noted that the dampers produce a substantial energy dissipation, while also change the behavior of a structure from a moment resisting frame to a braced frame. Their effect in the structure cannot be assessed from their individual mechanical properties only. A complete analytical
model of the super-structure including also the above devices is necessary for the overall system evaluation (see Section 5).
SECTION 4

EXPERIMENTAL STUDY OF RETROFITTED STRUCTURE

EARTHQUAKE SIMULATOR TESTING

4.1 Retrofit of Damaged Reinforced Concrete Model

A three story 1:3 scale model structure with lightly reinforced concrete frames, damaged by prior testing with moderate and severe earthquake (Bracci et al. 1992a, 1992b) was retrofitted by conventional concrete jacketing of interior columns and joint beam enhancements and was damaged again by several severe earthquakes (Bracci et al. 1992c). The same structure was further used to assess the possibility of retrofit of damaged frames with supplemental dampers installed in braces attached to the concrete joints. The study was developed to assess efficiency and structural interaction of various types of dampers, i.e.:

(a) viscoelastic dampers of 3M Company (Lobo et al. 1993, Shen et al. 1993).

(b) fluid viscous damper of Taylor Devices Inc. (Reinhorn et al. 1995)a.

(c) friction dampers of Tekton Co. and Sumitomo Co. (This report).

(d) viscous walls of Sumitumo Construction Co. (Reinhorn et al. 1995)b.

The objectives of the retrofit was (a) to reduce overall damage progression in severe episodes of earthquakes; (b) to provide data for analytical modeling of inelastic
structures equipped with dampers of hysteretic behavior and (c) to determine the force transfer in the retrofitted structures and its local effects.

The description of the model, the supplemental dampers and the testing program are described in this section.

4.2 Structure Model for Shaking Table Study

The structure was a three story 1:3 scale reinforced concrete frame structure original only for gravity loads without any special seismic provisions. The model was scaled from a prototype using mass simulation (Bracci et al. 1992a) The structural model had a floor weight of 120 kN (27,000 lbs). The structure had 50.8 mm (2 in) thick slabs supported by 76.2x172.4 mm (3x6 in) beams supported by 101.6x101.6 mm (4x4 in) columns before retrofit (see Fig. 4-1 and 4-2). After the conventional retrofit the interior columns were increased to 152.4x152.4 mm (6x6 in) by concrete jacketing with longitudinal post-tensioned reinforcement and with a column capital at each floor obtained by a fillet of joint connection (see Fig. 4-3 and 4-4).

The columns were symmetrically reinforced using 1.2%, total reinforcement ratio, and the beams had 0.8% positive reinforcement along entire beam and 0.8% negative reinforcement ratio above the supports. Detail of reinforcement and material properties can be found in Bracci et al. 1992a. A summary of this information is included in Appendix A for sake of completion.
Figure 4-1  Perspective View of 1:3 scale R/C Frame Structure

a. Before Conventional Retrofit

b. After Conventional Retrofit of Columns
Figure 4-2  Building Dimensions and Location of Local Measuring Devices in Columns
Figure 4-3 Conventional Retrofit by Jacketing of Interior Columns
Section 1

Figure 4-4  Detail of Conventional Retrofit with Concrete Jacketing and Joint Fillet
### Table 4-1a Moment Capacities of Structural Sections (units kips in)

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<tr>
<th></th>
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<tr>
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<td>Interior</td>
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<tr>
<td></td>
<td>(1)</td>
<td>(2)</td>
</tr>
<tr>
<td>Original Structure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3rd floor</td>
<td>Top</td>
<td>22</td>
</tr>
<tr>
<td>Bottom</td>
<td>22</td>
<td>0.01900</td>
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<tr>
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<td>Top</td>
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<td>Bottom</td>
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<td>0.01400</td>
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<td>Bottom</td>
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<td>0.01100</td>
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<td>After Conventional Retrofit</td>
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<tr>
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<td>Bottom</td>
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<tr>
<td>Bottom</td>
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<td>0.00048</td>
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<tr>
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<tr>
<td>Bottom</td>
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<td>0.00041</td>
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</table>

1 kips = 4.45 kN, 1 in = 25.4 mm.

### Table 4-1b Shear Capacities of Structural Sections (units kips)

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<td>1st floor</td>
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<td>After Conventional Retrofit</td>
<td></td>
<td></td>
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<tr>
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<td>0.800</td>
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<td>2nd floor</td>
<td>5.770</td>
<td>0.978</td>
</tr>
<tr>
<td>1st floor</td>
<td>5.770</td>
<td>1.244</td>
</tr>
</tbody>
</table>

1 kips = 4.45 kN
The moment and shear capacities of the sections before and after retrofit are listed in Table 4-1a and 4-1b. The moment capacities were calculated based on data in the Appendix A. It should be noted that the cracking and yielding of a section reduce the moment of inertia of sections and therefore only a fraction of the gross stiffness is active during a seismic event (Bracci et al. 1992b).

The structure was subjected to earthquake simulated motion using the shaking table at University of Buffalo. Moderate (peak ground acceleration PGA 0.2g) and severe episodes (PGA=0.3g) were used to verify the seismic behavior and the efficiency of structure suffered damage near collapse (90%, based on a damage index normalized to a unit which means collapse), the conventionally retrofitted structure suffered less damage, in repairable range. The original structure displayed a soft-column-side-sway mechanism. The conventionally retrofitted structure developed a safer beam-side-sway mechanism, which explains the reduced damage.

However, the structure developed inelastic behavior and damage. Therefore the structure was further retrofitted as presented in the next section.

4.3 Retrofit with Supplemental Friction Dampers

The structure was retrofitted with additional damping braces in the middle bay of each frame at all floors as shown in Fig. 4-5 and 4-6. The structure was also retrofitted with additional damping braces on 1st and 2nd floor only and retrofitted with additional damping braces on 1st floor only. The different configurations are shown in Fig. 4-7. In the following contents, the model with friction dampers or the model with 6 friction
Figure 4-5  Perspective View of the Frame with Installed Damping Devices
Figure 4-6 Location of Dampers and Measuring Devices
(a) Configuration of the model with 6 dampers (2 dampers each floor)

(b) Configuration of the model with 4 dampers (2 dampers each floor, 1st and 2nd floor only)

(c) Configuration of the model with 2 dampers at first floor only

Fig. 4-7 Different configurations of the tested model
Figure 4-8  Perspective View of Tekton Fricitn Dampers Installed in the Mid-bay of the Frame

Figure 4-9  Installation Detail of a Tekton Fricitn Damper Installed in the Mid-bay of the Frame
Figure 4-10  Perspective View of Sumitomo Friction Dampers Installed in the Mid-bay of the Frame

Figure 4-11  Installation Detail of a Sumitomo Friction Damper Installed in the Mid-bay of the Frame
dampers refers to configuration (a) in Fig. 4-7. The model with 4 dampers refers to configuration (b), and the model with 2 dampers refers to configuration (c). The details of the braces are shown in Fig. 4-8 and 4-9 for Tekton devices and Fig. 4-10 and 4-11 for Sumitomo devices.

The braces were connected to the floors at base and top of columns and transferred loads to the joint through the beams and the fillet joint (see Fig. 4-9 and 4-11). The braces consist of an A36 L6x6x1/2" steel angle connected through 1/2 in diameter bolts to allow for a pinned connection at its ends.

4.3.1 Friction Dampers

The dampers installed in the brace were specially designed by Tekton Company and Sumitomo Company for the structure model as shown in Fig. 2-1 and 2-2. The structure model incorporated with Tekton friction dampers was tested after the model incorporated with Sumitomo friction dampers. The friction force of the dampers were calibrated to about 3.2 kips. The damper was connected to the brace using a load cell with a capacity of 30,000 lbs. The Tekton friction dampers (presented in Section 2) were installed in the structure as follows: #4 and #2 at first floor, #1 and #6 at second floor, and #3 and #5 at third floor, where the first ones in the pairs indicate east frame of the structure (see damper properties in Table 2-1), and the similar arrangement of Sumitomo friction dampers can be seen in Table 2-1. Efficiency of using dampers was investigated by using dampers on lower floor only.
The damper construction can prevent rotations between its two ends which is suitable to prevent buckling in the brace assembly.

### 4.4 Instrumentation

The structure was instrumented with motion and force transducers to monitor the force transfer within the structure. A series of accelerometers were installed horizontally at each floor and at its base. Five directional load cells measuring axial loads, shear forces in two directions, bending moments in two directions were installed in the mid-height of each column of east frame at first and second floor (see Fig. 4-2). For detailed description of load cells see Bracci et al. 1992a. The braces were instrumented with an axial load cell and a longitudinal displacement transducer (see Fig. 4-6) to measure the movement in the damper.

The structure was placed on the shaking table at SUNY/Buffalo. The shaking system was monitored for displacements, velocities and accelerations in horizontal, vertical and rocking directions. A total of 83 channels of data were recorded during each earthquake.

The instrumentation consisting of load cells, displacement transducers and accelerometers is detailed in APPENDIX B with a list of monitored channels and their corresponding descriptions are given in Table 4-2. A total of 83 channels were monitored.
Table 4-2 List of Channels (with reference to Fig. B-1)

<table>
<thead>
<tr>
<th>CHANNEL</th>
<th>NOTATION</th>
<th>INSTRUMENT</th>
<th>RESPONSE MEASURED</th>
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<tbody>
<tr>
<td>1</td>
<td>AH1</td>
<td>ACCEL</td>
<td>Longitudinal accel. - on the base, east side</td>
</tr>
<tr>
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<td>ACCEL</td>
<td>Longitudinal accel. - on the base, west side</td>
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<td>AH3</td>
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<td>Longitudinal accel. - 1st floor, east side</td>
</tr>
<tr>
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<td>AH4</td>
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</tr>
<tr>
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<td>AH5</td>
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<tr>
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</tr>
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</tr>
<tr>
<td>15</td>
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ACCEL= Accelerometer, DT= Displacement Transducer; Longitudinal = North-South Direction
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<td>DT</td>
<td>Lateral displacement on shaking table</td>
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<td>ALAT</td>
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<td>Vertical acceleration on shaking table</td>
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ACCEL= Accelerometer, DT= Displacement Transducer.
4.5 Experimental Program

The study was performed using simulated ground motion of two types: (i) white noise excitations in horizontal direction to identify structural properties of the structure at various stages of testing and to verify functionality of instrumentation; different levels of white noise excitations were used to identify structural properties when dampers were at stick state and at sliding state, and (ii) various levels of simulated historical earthquakes scaled to produce elastic and inelastic response in the structure. The structure was tested with and without dampers for comparison sakes. The testing schedule is presented in Table 4-3a and 4-3b. The tests without dampers (tests #32 through #48) were done at lower maximum levels than the tests with dampers, to permit further repairing and testing (without necessity to repair extensive damage).

A total of 28 earthquake simulation tests were performed for the structure model with six Tekton friction dampers (two each floor), with four Tekton friction dampers (two each floor at first and second floor), with two Tekton friction dampers at first floor and bare frame. Nine earthquake simulation tests were performed for the structure model with six Sumitomo friction dampers (two each floor). The simulated ground motion included Taft N21E 1952, El-Centro S00E 1940, Hachinohe 1964, Pacoima Dam S16E 1971, and Mexico City N90E 1985. The tests were performed using the horizontal components only. The simulated requirements for a 1:3 scale structure using artificial mass simulation dictated a reduction of the time interval for the horizontal accelerogram of $1: \sqrt{3}$. The
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Notes: 1. pretest; 2. Setup problem of braces (transverse vibration); 3. Connection problem of Teposonic on dampers.
Table 4-3b Shaking Table Experimental Program - Tekton Friction Dampers

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Notes:  1. two dampers each floor; 2. two dampers each floor for 1st and 2nd floors; 3. two dampers only at 1st floor.
acceleration, displacement and velocities and response spectra of the shaking table simulated motion are shown in Fig. 4-12 through 4-21.

4.6 Identification of Structure Properties

A few levels (0.025g, 0.1g and 0.15g) of narrow band (0-25) white noise excitations were used to shake the structure in order to identify initial stiffness of structure before and after each severe shaking. The low level dynamic properties, periods and mode shapes were determined as described below.

4.6.1 Experimental Identification of Dynamic Characteristics of Model

The structure is assumed to behave linearly elastic at low amplitude levels. The increased structural response is therefore:

\[ \ddot{U}_j(\omega) = \left( \sum_{j=1}^{N} \phi_j H_j(\omega) \Gamma_j \right) \ddot{U}_g(\omega) \] .................................................................(4-1)

where \( \ddot{U}_j(\omega), \ddot{U}_g(\omega) \) indicate the Fourier transforms of the absolute acceleration response (at d.o.f i) and the base excitation, respectively, \( H_j(\omega) \) indicates the complex frequency absolute acceleration response function:

\[ H_j(\omega) = \frac{r_j^2 + 2\xi_j r_j i}{(1 - r_j^2) + 2\xi_j r_j i} \] .................................................................(4-2)
Figure 4-12  Simulated Ground Motion El-Centro S00E Scaled to PGA 0.3g (t_m = t_p/\sqrt{3})
Figure 4-13  Elastic Response Spectra of Simulated El-Centro Earthquake PGA 0.3g
Figure 4-14  Simulated Ground Motion Taft N21E Scaled to PGA 0.2g ($t_m = t_p / \sqrt{3}$)
Figure 4-15  Elastic Response Spectra of Simulated Taft N21E Earthquake PGA 0.3g
Figure 4-16  Simulated Ground Motion Mexico City N90W Scaled to PGA 0.2g \( (t_m = t_p/\sqrt{3}) \)
Figure 4-17  Elastic Response Spectra of Simulated Mexico City Earthquake PGA 0.2g
simulated ground acceleration

Hachinohe 0.3g

simulated ground velocity

simulated ground displacement

Figure 4-18  Simulated Ground Motion Hachinohe N00S Scaled to PGA 0.3g
Figure 4-19  Elastic Response Spectra of Simulated Hachinohe N00S Earthquake PGA 0.3g

4-30
Figure 4-20  Simulated Ground Motion Pacoima S16E Scaled to PGA 0.3g
Figure 4-21 Elastic Response Spectra of Simulated Pacoima S16E Earthquake PGA 0.3g
where \( r_j = \frac{\omega_j}{\omega_0} \) is the model frequency ratio for mode \( j \), and \( i = \sqrt{-1} \). In Eq. (4-1) \( \phi_{ij} \) is the \( j \)-th mass (m) normalized mode shape for the \( i \)-th floor (DOF) satisfying the condition,

\[
\sum_{i=1}^{N} \phi_{ij}^2 m_i = 1, \quad \text{for } j = 1, N \quad \text{........................................................................(4-3)}
\]

and \( \Gamma_j \) is the modal participation factor:

\[
\Gamma_j = \sum_{i=1}^{N} \phi_{ij} m_i \quad \text{........................................................................(4-4)}
\]

For well separated modes, as obtained in the response of this structure, the acceleration response transfer function, which is defined as:

\[
T_{ai}(\omega) = \frac{\ddot{U}_{i}(\omega)}{\ddot{U}_{g}(\omega)} \quad \text{........................................................................(4-5)}
\]

is obtained at a resonant peak from single mode, \( k \), contribution from Eq. (4-1) for \( (H_j(\omega_k) \to 0, \quad \text{for } \omega_k \neq \omega_j ) \):

\[
T_{ai}(\omega) = \phi_{ik} H_k(\omega_k) \Gamma_k \quad \text{........................................................................(4-6)}
\]

The ratio of modal shapes are obtained from ratio of transfer functions from Eq. (4-6):

\[
\frac{\phi_{ik}}{\phi_{jk}} = \frac{T_{ai}(\omega_k)}{T_{aj}(\omega_k)} \quad \text{........................................................................(4-7)}
\]
At the peak obtained for frequency $\omega_k$, the absolute value of the complex frequency response function from Eq. (4-2) for $r_k = 1$ is obtained as:

$$|H_k(\omega_k)| = \frac{\sqrt{1 + 4\xi_k^2}}{2\xi_k}$$ ................................................................. (4-8)

Combining Eq. (4-6) and (4-8) the damping ratio $\xi_k$ can be derived:

$$\xi_k = \left(2\sqrt{\frac{T_{nl}(\omega_k)}{\phi_{ik}^T\Gamma_k}} - 1\right)^{-1}$$ ........................................................................ (4-9)

The damping ratio can be obtained from a recording at any degree of freedom $i$.

From the identification above, using the orthogonality conditions, the stiffness matrix of the structure can be obtained:

$$K = M\Phi_n^T\Omega\Phi_n^T M$$ .......................................................................................... (4-10)

in which $M$ is the mass matrix and $\Omega$ is:

$$\Omega = \text{diag}(\omega_1^2, \omega_2^2, \ldots, \omega_n^2)$$

while $\Phi_n$ is the mass normalized modal shapes matrix obtained identification using Eq. (4-7) and (4-3) ($\Phi^T M \Phi = I$). The system matrices can be reduced to $m \times m$, if only $m$ modes are retained in the analysis.

4-34
Assuming that the damping matrix also satisfies the orthogonality conditions, it can be expressed as:

\[ C = \mathbf{M} \Phi \zeta \Phi^T \mathbf{M} \]

where the modal damping matrix \( \zeta \) is:

\[ \zeta = \text{diag} [2 \xi_1 \omega_1, 2 \xi_2 \omega_2, \ldots, 2 \xi_n \omega_n] \]

\( \xi_i \) = i-th mode damping ratio

\( \omega_i \) = i-th natural frequency (rad/sec)

where \( \xi_i \) are the damping ratio obtained from Eq. (4-9) for each mode \( k \) with a modal frequency \( \omega_k \).

At high level of excitation the structure becomes inelastic and the above properties cannot be obtained. However, as an indicator of structure changes the "equivalent" dynamic properties can be defined in a similar manner using Eq. (4-7), (4-9) and (4-12) with the data obtained from the pseudo-transfer function, \( PT_{ti}(\omega) \), calculated from Eq. (4-5). It should be noted that while Fourier Transform of the excitation \( \hat{U}_g(\omega) \) remains constant during the response, the Fourier Transfer of the response \( \hat{U}_r(\omega) \) is only a "form of an average" of the inelastic response depending on the length of the record. The dynamic properties for the severe shaking were determined according to the above, as an indicator of the response.
4.6.2 Dynamic Characteristics of Structure

The dynamic characteristics of the structure were determined by the aforementioned identification method as indicated by the results in this section.

4.6.2.1 Structure without Supplementary Dampers

The story transfer functions of structure without dampers have small damping and well separated modes (see Fig. 4-22). The peaks occur precisely at the natural frequencies of the model are identified from low level white noise tests as following:

\[ f = \begin{bmatrix} 1.56 \\ 7.03 \\ 14.06 \end{bmatrix} \text{ (Hz)} \]

The mode shape matrix

\[
\Phi = \begin{bmatrix}
1.00 & -0.79 & -0.55 \\
0.84 & 0.36 & 1.00 \\
0.48 & 1.00 & -0.79
\end{bmatrix}
\]

\[
\text{or mass normalized } = \begin{bmatrix}
2.72 & -2.25 & -1.50 \\
2.28 & 1.03 & 2.72 \\
1.30 & 2.85 & -2.15
\end{bmatrix}
\]

Thus the stiffness matrix can be calculated from Eq. 4-10 as following:

\[
K = \begin{bmatrix}
137.92 & -175.26 & 63.69 \\
-175.26 & 295.51 & -194.17 \\
63.69 & -194.17 & 255.21
\end{bmatrix}
\]
Figure 4-22 Transfer Function from White Noise Ground Motion (with and w/o Tekton Friction Dampers)
### 4.6.2.2 Structure with Supplementary Dampers

The structure was stiffened significantly after the friction damping devices were installed. From the transform function of white noise excitation (Fig. 4-22 and 4-23), the nature frequencies of the model with friction dampers can be identified as follows:

For the structure with Tekton friction dampers:

\[
\begin{bmatrix}
3.33 \\
11.00 \\
17.30
\end{bmatrix}
\] (Hz)

and the mode shape matrix

\[
\begin{bmatrix}
1.00 & -0.78 & -0.47 \\
0.75 & 0.52 & 1.00 \\
0.39 & 1.00 & -0.43
\end{bmatrix}
\]

Thus the stiffness matrix can be calculated by Eq. 4-10 as following

\[
\begin{bmatrix}
256.48 & -335.81 & -12.34 \\
-335.81 & 647.61 & -155.92 \\
-12.34 & -155.92 & 288.53
\end{bmatrix}
\]

For the structure with Sumitomo dampers:

\[
\begin{bmatrix}
3.32 \\
11.30 \\
17.60
\end{bmatrix}
\] (Hz)
Figure 4-23  Transfer Function from White Noise Ground Motion (with and w/o Sumitomo Friction Dampers)
and the mode shape matrix

\[
\Phi = \begin{bmatrix}
1.00 & -0.74 & -0.54 \\
0.78 & 0.49 & 1.00 \\
0.39 & 1.00 & -0.90
\end{bmatrix}
\]

or mass normalized \[
\begin{bmatrix}
2.85 & -2.09 & -1.01 \\
2.22 & 1.39 & 2.61 \\
1.11 & 2.83 & -2.35
\end{bmatrix}
\]

Thus the stiffness matrix can be calculated by Eq. 4-10 as following

\[
K = \begin{bmatrix}
186.34 & -216.23 & 2.87 \\
-216.23 & 466.42 & -265.12 \\
2.87 & -265.12 & 531.37
\end{bmatrix}
\]

A summary of the dynamic characteristics of the structure derived from the severe shaking (see Fig. 4-24 and 4-25) is presented in Table 4-4. It should be noted that the fundamental period of the structure at low level of shaking is reduced significantly when dampers are installed, which indicates that the braces and the dampers stiffen the structure. In fact the moment resisting frame becomes a braced frame at low deformations, before the devices slip, however, the apparent period of the structure during severe shaking is 130% larger than at the low level shaking. This can be attributed to the frequent slip of dampers and the softening effect during the inelastic response of the structure.

The damping increases at and severe shaking approximately 5 times, but increases little at low amplitude shaking. The increase in damping at severe shaking is attributed in part to the inelastic response of structure and in part due to the increase in energy dissipation at lower amplitude in the added dampers.
Figure 4-24  Transfer Function from El-Centro 0.3g Ground Motion (with and w/o Tekton Friction Dampers)
Figure 4-25  Transfer Function from El-Centro 0.3g Ground Motion (with and w/o Sumitomo Friction Dampers)
### Table 4-4 Dynamic Characteristics of the Structure

<table>
<thead>
<tr>
<th>Ground motion</th>
<th>PGA(g’s)</th>
<th>Damping (% of critical)</th>
<th>Fundamental period / Frequency (second) / (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Low amplitude testing</td>
<td>Strong motion testing</td>
</tr>
<tr>
<td></td>
<td>(1)</td>
<td>(2)</td>
<td>(3)</td>
</tr>
<tr>
<td>Without dampers</td>
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<td></td>
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<tr>
<td>El-Centro S00E</td>
<td>0.3</td>
<td>3</td>
<td>6</td>
</tr>
<tr>
<td>Taft N21E</td>
<td>0.2</td>
<td>3</td>
<td>5</td>
</tr>
<tr>
<td>With Fluid Dampers (Taylor)</td>
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<tr>
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<td>0.3</td>
<td>16</td>
<td>28</td>
</tr>
<tr>
<td>Taft N21E</td>
<td>0.2</td>
<td>16</td>
<td>26</td>
</tr>
<tr>
<td>With Friction Dampers (Sumitomo)</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>El-Centro S00E</td>
<td>0.3</td>
<td>7</td>
<td>23</td>
</tr>
<tr>
<td>Taft N21E</td>
<td>0.2</td>
<td>7</td>
<td>26</td>
</tr>
<tr>
<td>With Viscous Damping Walls</td>
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<tr>
<td>El-Centro S00E</td>
<td>0.3</td>
<td>50</td>
<td>46</td>
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<tr>
<td>Taft N21E</td>
<td>0.2</td>
<td>49</td>
<td>47</td>
</tr>
</tbody>
</table>

Low amplitude testing - white noise testing before and after simulated ground motion testing.
Strong motion testing - simulated ground motion testing indicated in column (2).
Approximated analytically - according to Lobo et al., 1993.
The equivalent modal damping $\xi_{TOT,k}$ can be estimated for a mode $k$ according to Lobo et al. (1993):

$$\xi_{TOT,k} = \Delta \xi_k + \xi_k \left( 1 - \alpha_i + \alpha_i^2 - \alpha_i^3 + \cdots \right)$$

where $\Delta \xi_k$ is the damping increase due to added damping devices:

$$\Delta \xi_k = \frac{1}{2\omega_k} \left( \Phi_k^T \Delta C \Phi_k \right)$$

or simply:

$$\Delta \xi_k = \frac{1}{2\omega_k} \sum_i c_i \left( \phi_{ik} - \phi_{i-1,k} \right)^2 \cos \theta_i$$

while $\xi_k$ is the original damping the structure without dampers and

$$\alpha_k = \frac{1}{\omega_k^2} \sum \Delta k_i \left( \phi_{ik} - \phi_{i-1,k} \right)^2 \cos^2 \theta_i$$

and $c_i$ is the equivalent damping constant of friction dampers at $i$th degree of freedom. The equivalent damping constant $c_i$ of friction damping device can be determined by matching the energy dissipation of the friction device with viscous damping device. Assume the slip force of the friction damper is $F_{sy}$ and the spectral displacement in the damping device is $S_d$. For any test of viscous damper at a frequency $\Omega$ around structural fundamental frequency and an amplitude $u_0$, if the area included in one hysteretic loop $W_d$ (energy
dissipated in one cycle) is equal to $F_y S_d$, we can calculate the equivalent damping constant as:

$$c_i = \frac{F_y S_d}{\pi \Omega u_0^2} \quad \text{(4-14)}$$

where $\Phi_k$ and $\omega_k$ are the vector $k$ in the modal shapes matrix and the frequency for the undamped structure, respectively.

The approximated values calculated according to the above are listed in Table 4-4 to capture damping increase in the severe shaking.

4.7 Seismic Response

The experimental results of the model without dampers and with different dampers configurations demonstrate clearly the benefits provided by friction damping devices. The comparisons of time history response of structure model with and without dampers are shown in Fig. 26 to 4-33 (El-Centro 0.3g ground motion). Time histories responses of structure model under other ground motions are presented in Fig. 4-44 to 4-51 for reference. The peak response at various levels of shaking is summarized in Table 4-5a and 4-5b. The forces in the structural components are shown in Fig. 4-52. The efficiency of using dampers only in lower floors can be easily seen for the tested model, but further detailed consideration should be taken for different structures. As can be seen in the test with two dampers at first floor only, the drift at second floor may be larger than that at first floor (Taft 0.2g) or close to that at first floor (El-Center 0.3g test). The shear force at
<table>
<thead>
<tr>
<th>Model</th>
<th>PGA target achieve (g's)</th>
<th>story drift 1st (in)</th>
<th>story drift 2nd (in)</th>
<th>story drift 3rd (in)</th>
<th>story shear 1st (kips)</th>
<th>story shear 2nd (kips)</th>
<th>story shear 3rd (kips)</th>
<th>all column shear (kips)</th>
<th>single damper force (kips)</th>
<th>single interior col. shear (kips)</th>
<th>single exterior col. shear (kips)</th>
<th>single interior col. axial force (kips)</th>
<th>single exterior col. axial force (kips)</th>
<th>first story response</th>
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<td>0.14 0.12</td>
<td>0.07 0.07</td>
<td>0.31 0.31</td>
<td>4.23 3.31 2.14</td>
<td>1.32 0.72</td>
<td>0.57 0.73</td>
<td>2.14</td>
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<td>3.34 1.05</td>
<td>2.03 2.50</td>
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<td></td>
<td>0.20 0.21</td>
<td>0.69 0.42</td>
<td>0.23 0.23</td>
<td>1.27 14.82</td>
<td>11.02 7.00</td>
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<td>1.05</td>
<td>1.05</td>
<td>1.05 1.05</td>
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<tr>
<td>With 6 friction</td>
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<td>1.05</td>
<td>2.63 2.63</td>
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<td>First story drift response</td>
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<tr>
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<td>0.21 0.17</td>
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<td>0.44 12.56</td>
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<td>9.90 7.20</td>
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<td>3.93 13.98</td>
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<tr>
<td>Model</td>
<td>PGA units (g's)</td>
<td>1st (in)</td>
<td>2nd (in)</td>
<td>3rd (in)</td>
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<td>Single interior col. shear (kips)</td>
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*the stroke limit of the damper was exceeded.
Figure 4-26  Comparison of Displacement Response History for Structure without and with Six Tekton Friction Dampers, from El-Centro Earthquake PGA 0.3g Test
Figure 4-27 Comparison of Acceleration Response History for Structure without and with Six Tekton Friction Dampers, from El-Centro Earthquake PGA 0.3g Test
Figure 4-28  Comparison of Displacement Response History for Structure without and with Six Sumitomo Friction Dampers, from El-Centro Earthquake PGA 0.3g Test
Figure 4-29  Comparison of Acceleration Response History for Structure without and with Six Sumitomo Dampers, from El-Centro Earthquake PGA 0.3g Test
Figure 4-30  Comparison of Displacement Response History for Structure without and with Four Tekton Friction Dampers, from El-Centro Earthquake PGA 0.3g Test
Figure 4-31  Comparison of Acceleration Response History for Structure without and with Four Tekton Friction Dampers, from El-Centro Earthquake PGA 0.3g Test
Figure 4-32  Comparison of Displacement Response History for Structure without and with Two Tekton Friction Dampers, from El-Centro Earthquake PGA 0.3g Test
Figure 4-33  Comparison of Acceleration Response History for Structure without and with Two Tekton Friction Dampers, from El-Centro Earthquake PGA 0.3g Test
Figure 4-34  Displacement Time History Response of the Model with Six Tekton Friction Dampers, Taft Earthquake PGA 0.2g
Figure 4-35  Acceleration Time History Response of the Model with Six Tekton Friction Dampers, Taft Earthquake PGA 0.2g
Figure 4-36  First Floor Single Damper Response, Taft Earthquake PGA 0.2g
Figure 4-37 Displacement Time History Response of the Model with Six Tekton Friction Dampers, Taft Earthquake PGA 0.4g
Figure 4-38  Acceleration Time History Response of the Model with Six Tekton Friction Dampers, Taft Earthquake PGA 0.4g
Figure 4-39  First Floor Single Damper Response, Taft Earthquake PGA 0.4g
Figure 4-40  Displacement Time History Response of the Model with Six Tekton Friction Dampers, Mexico City Earthquake PGA 0.1g
Figure 4-41  Acceleration Time History Response of the Model with Six Tekton Friction Dampers, Mexico City Earthquake PGA 0.1g
Figure 4-42  First Floor Single Damper Response, Mexico City Earthquake PGA 0.1g
Figure 4-43  Displacement Time History Response of the Model with Six Tekton Friction Dampers, Mexico City Earthquake PGA 0.2g

4-65
Figure 4-44  Acceleration Time History Response of the Model with Six Tekton Friction Dampers, Mexico City Earthquake PGA 0.2g
Figure 4-45   First Floor Single Damper Response, Mexico City Earthquake PGA 0.2g
Figure 4-46 Displacement Time History Response of the Model with Six Tekton Friction Dampers, Hachinohe Earthquake PGA 0.3g
Figure 4-47  Acceleration Time History Response of the Model with Six Tekton Friction Dampers, Hachinohe Earthquake PGA 0.3g
Figure 4-48  First Floor Single Damper Response, Hachinohe Earthquake PGA 0.3g
Figure 4-49  Displacement Time History Response of the Model with Six Tekton Friction Dampers, Pacoima Earthquake PGA 0.3g
Figure 4-50  Acceleration Time History Response of the Model with Six Tekton Friction Dampers, Pacoima Earthquake PGA 0.3g
Figure 4-51  First Floor Single Damper Response, Pacoima Earthquake PGA 0.3g
Figure 4-52  Forces in Structural Components at First Floor, from El-Centro PGA 0.3g Test
(a) and (d) without Dampers; (b), (c) and (e) with Six Tekton Dampers
the second floor remains almost the same as without dampers. The distribution for forces during the response shows that while the overall response is improved, the local response may worsen or remain unaffected. It should be noted that while the deformations are substantially reduced at all floors for same level of excitation, the total base shear is only minimally influenced. The overall shear forces at severe excitation remain at the same level with minor increase. When the peak ground acceleration is increased, except for the Mexico City record, the long period excitation with high velocity maintains the internal forces increase for a longer time resulting in longer deformations and shear forces.

The friction damping devices seem to have limited influence where a monotonically increasing acceleration is predominant in the record, in particular if this increase has a long duration. Evaluations for such excitations are records obtained on soft soil (Mexico City 1985, Buchanest 1977, etc.) or linear fault records. For such cases, a different damping system may be required.

For a single ground record, the increase of peak ground acceleration (see Taft N21E experiment) produce a non-proportional increase in the displacement response. The damped response for 0.40g PGA is, however, smaller than the undamped response for 0.20g PGA.

Using damper at the first two floors produce better response at bottom floor and less at the top. The dampers have immediate influence on the local floor response. Similar response is obtained in using dampers at first floor only. The story forces are influenced also locally by the capacity diagram at each floor and by the local spectral demand (see
Section 6.2). This can be observed also from the typical time history responses in Fig. 4-30 to 4-33. While the total displacements are reduced at all floors the peak story absolute accelerations are not reduced, moreover, are increased at the top floor. The total energy balance (see Section 1, Eq. (1-1)) obtained from experimental data is displayed in Fig. 4-53 for $E_r = \int_0^t m(\dddot{u} + \dddot{u}_s) du_s$; $E_k = \frac{1}{2} m(\dddot{u} + \dddot{u}_s)^2$; $E_s = \frac{1}{2} ku^2$. While the total energy input is increased due to stiffening of the structure, the internal energy is redistributed such that 80% to 90% is taken by the supplemental dampers and dissipated, while hysteretic energy dissipation demand is reduced 85% to 95% in presence of dampers. The reduction of the demand for hysteretic energy dissipation is particularly important since it is preventing further deterioration of columns.

While the total shear forces at the base of the damped structure are increased in the presence of dampers, the force in the individual columns are smaller than in the undamped case. This indicates that the forces are partially transferred through the braces that protect the original columns (Table 4-5 and Fig. 4-52).

The axial force fluctuation is larger in the column (see Fig. 4-54), but not large enough to influence the flexural capacity of the columns (see Fig. 4-55). In taller structures, this is an important issue since the axial force may accumulate if a single bay of frame is braced. However, a proper redistribution of braces can eliminate or reduce the concentration and accumulation effects.
Figure 4-53  Energy Distribution in Structure w/o and with Six Tekton Friction Dampers
Figure 4-54 Axial Force Fluctuation in First Floor Interior Column for Simulated Earthquake El-Centro 0.3g. (a) w/o Dampers; (b) with Six Tekton Friction Dampers
Figure 4-55  Forces in Column vs Structural Capacity for El-Centro PGA 0.3g
(a) without Dampers, (b) with Six Tekton Friction Dampers
4.8 Summary of the Experimental Study

The experiment indicated that the dampers show a small stiffness increase depending on the intensity of the earthquake and influence control deformation through damping. However, the forces transmitted to the foundation and the structure's accelerations are only minimally reduced and in some cases minimally increased. The energy dissipation capacity increase with the increase of intensity of the earthquake and the period of the structure varies with the intensity of the earthquake which will prevent possible resonant. The main benefit of the dampers in such inelastic structures consists in transferring of the energy dissipation needs from the columns to the dampers while controlling the lateral drifts and deformations. These results should be expected in all inelastic structures, as shown further by the analytical study and the approximated analyses.
SECTION 5

MODELING OF INELASTIC STRUCTURES WITH SUPPLEMENTAL DAMPERS

5.1 Modeling of Inelastic Structures

Inelastic analysis of structures to wind and earthquake loading is usually performed using step-by-step integration of equations of motion, which are representative to structures with variable stiffness due to cracking yielding, deterioration and secondary effects.

In this study the structure is modeled as a structural frame made of rigidly or semi-rigidly connected columns, beams, shear walls and braces (see Kunnath et al. 1992, Reinhorn et al. 1994). The structural members are modeled as macro-models with inelastic properties described by: (i) an extensive hysteretic model with stiffness and strength deterioration and pinching due to crack opening and closing (see Fig. 5-1); (ii) a non-symmetric distributed plasticity model obtained through a distributed flexibility model (see Fig. 5-2). The structure is modeled by the matrix equation:

\[
M\ddot{u} + C\dot{u} + R(u) = -M\ddot{u}_g + F_w
\]

where \( u \), \( \dot{u} \), \( \ddot{u} \) are the time dependent response, vector of displacement, velocity and acceleration respectively, \( \ddot{u}_g \) is the ground acceleration; \( F_w \) is the wind force vector. \( M \) is the mass matrix, \( C \) is the inherent damping matrix of structure and \( R \) is the nonlinear
resistance vector of the structure obtained from the addition of individual component’s resistance. The resistance vector is a function of deformation based on models shown in Fig. 5-1 and 5-2 (Reinhorn et al. 1994).

The equation of motion can be written in incremental form as:

\[ M\ddot{\Delta u} + C\Delta \dot{u} + K\Delta u = -M\Delta \ddot{u}_g + \Delta F_w \] ................................. (5-2)

where

\[ K = \frac{\Delta R(u)}{\Delta u} \] ................................. (5-3)

is the instantaneous stiffness assumed constant during a specific incremental computation time step.

5.2 Modeling of Structure with Supplemental Dampers

The structure with supplemental dampers will have another dissipation term in the structure’s equation:

\[ M\ddot{\mu} + C\dot{\mu} + R(u) + F_D(u) = -M\ddot{\mu}_g + F_w \] ................................. (5-4)

where the supplemental damping forces \( F_D \) obtained from suitable transformation of braces forces to the corresponding degrees of freedom.

\[ F_D(u) = DF_Di(u_i) \] ................................. (5-5)
One dimensional schematic of triaxial hysteretic beam column element.

Figure 5-1  an Extensive Hysteretic Model with Stiffness and Strength Deterioration and Pinching Due to Crack Opening and Closing

5-3
Figure 5-2  a Non-symmetric Distributed Plasticity Model Obtained through a Distributed Flexibility Model
where \( D \) is a location matrix, \( F_{D_i} \) is the vector of individual device forces, and \( u_i \) are the deformations and velocities of devices, \( i \).

### 5.2.1 Modeling Using Bouc-Wen’s Model

According to the discussion in Section 3.1.1, Bouc-Wen’s models offer solutions in time domain, if solved simultaneously with the rest of the structure. According to these models:

\[
F_D(u) = DF_{D_i}(u_i) \quad \text{(5-6)}
\]

where \( D \) is the location matrix and the damping force \( F_i \) in each damper \( i \) is given in a differential form for Bouc-Wen’s model:

\[
F_D = k_0 (\alpha U + (1-\alpha)ZU_y) \quad \text{repeat(3-1)}
\]

\( Z \) is a nondimensional quantity given by:

\[
\dot{Z} = \left( \frac{\dot{U}}{U_y} \right) \left[ A - Z^n \left[ \gamma \, \text{sgn}(\dot{U}Z) + \beta \right] \right] \quad \text{repeat(3-2)}
\]

The solution for models represented by differential forces is presented below.

### 5.2.2 Solution of Differential Equations

The solution is thought for the equations in incremental form:

\[
M\Delta \dddot{u} + C\Delta \dddot{u} + K\Delta u + D\Delta F_D = -MI\dddot{u}_z + F_w \quad \text{(5-7)}
\]
in which the incremental force, $\Delta F_D$ can be calculated using the semi-implicit Runge-Kutta method (Rosenbrook 1964):

$$
\Delta F_{Dk} = R_1 k_k + R_2 l_k 
$$

(5-8)

where $F_{Dk}$ and $F_{Dk-1}$ are the damper force at $k$-th and $(k-1)$-th time step, respectively. $k_k$ and $l_k$ are determined by solving following coupled equations:

$$
k_k = \Delta t \left[ f(F_k, u_k, \dot{u}_k)_{t-\Delta t} + a_1 \frac{\partial f(F_k, u_k, \dot{u}_k)_{t-\Delta t}}{\partial F} k_k \right] \quad \text{......................................(5-9a)}
$$

$$
l_k = \Delta t \left[ f(F_k + b_k k_k, u_k, \dot{u}_k)_{t-\Delta t} + a_2 \frac{\partial f(F_k + c_k k_k, u_k, \dot{u}_k)_{t-\Delta t}}{\partial F} l_k \right] \quad \text{......................................(5-9b)}
$$

or directly:

$$
k_k = \Delta t \left[ 1 - a_1 \Delta t \frac{\partial f(F_k, u_k, \dot{u}_k)_{t-\Delta t}}{\partial F} \right]^{-1} f(F_k, u_k, \dot{u}_k)_{t-\Delta t} \Delta t \quad \text{......................................(5-10a)}
$$

$$
l_k = \Delta t \left[ 1 - a_2 \Delta t \frac{\partial f(F_k + c_k k_k, u_k, \dot{u}_k)_{t-\Delta t}}{\partial F} \right]^{-1} f(F_k + b_k k_k, u_k, \dot{u}_k)_{t-\Delta t} \Delta t \quad \text{......................................(5-10b)}
$$

In above equations, the constant parameters $R_1$, $R_2$, $a_1$, $a_2$, $b_1$ and $c_1$ are obtained from the solution of the following equations:

$$
R_1 + R_2 = 1 \quad \text{......................................................(5-11a)}
$$
\[ R_1 a_1 + R_2 (a_2 + b_1) = \frac{1}{2} \] \hspace{1cm} (5-11)\text{b}

\[ R_1 a_1^2 + R_2 [a_2^2 + (a_1 + a_2)b_1] = \frac{1}{6} \] \hspace{1cm} (5-11)\text{c}

\[ R_2 \left( a_2 c_1 + \frac{1}{2} b_1^2 \right) = \frac{1}{6} \] \hspace{1cm} (5-11)\text{d}

In this study, a series of coefficients were selected (see Reinhorn et al. 1994) to obtain a fourth order truncation error \( O(\Delta t^4) \) that satisfy Eq. (5-11), and they are: \( R_1 = 0.75 \); \( R_2 = 0.25 \); \( a_1 = a_2 = 0.7886751 \); \( b_1 = -1.1547005 \) and \( c_1 = 0 \).

It should be noted that the incremental force \( \Delta F_i \) requires information about \( u, \dot{u} \) at the end of the incremental interval \( t+\Delta t \). Therefore several iterations are required to solve Eq. (5-7) and (5-8) simultaneously.

**5.2.3 Solution of Seismic Response of Structure**

The solution of the equations of motion can be obtained from the algorithm outlined in Table 5-1. The algorithm in Table 5-1 will provide the solution for Bouc-Wen's models (Section 5.2.1).

**5.2.4 Analytical Damage Evaluation**

The solution presented in the preceding section was incorporated in an analytical platform, IDARC Version 3.2 (Reinhorn et al. 1992). In this platform, the inelastic
Table 5-1 Numerical Solution Algorithm

A. Equations
\[ \Delta f_I + \Delta f_D + \Delta f_S + \Delta F_D = \Delta P \]
in which \[ \Delta f_I = M \Delta \ddot{u}; \Delta f_D = C \Delta \dot{u}; \Delta f_S = \tilde{K} \Delta u \] and
\[ F_D = \bar{k}_D[\alpha \dot{u} + (1 - \alpha)Z \ddot{u}] \]
\[ Z = \frac{(\dot{u} / \gamma_y)}{A - Z^\alpha[0\text{sgn}(\dot{u} Z) + \beta]} \]

B. Initial Condition
1. Form stiffness matrix \( \tilde{K} \), mass matrix \( M \), and damping matrix \( C \).
2. Initialize \( u_0 \) and \( \dot{u}_0 \).
3. Select time step \( \Delta t \), choose parameter \( \alpha = 0.25 \) and \( \delta = 0.5 \), calculate integration constants:
   \[ a_0 = \frac{1}{\alpha \Delta t^2}; a_1 = \frac{\delta}{\alpha \Delta t}; a_2 = \frac{1}{\alpha \Delta t}; a_3 = \frac{1}{2\alpha} - 1; \]
   \[ a_4 = \frac{\delta}{\alpha} - 1; a_5 = \frac{\Delta t}{2} \left( \frac{\delta}{\alpha} - 2 \right); a_6 = \Delta t(1 - \delta); a_7 = \delta\Delta t. \]
4. Form effective stiffness matrix \( K^* = \tilde{K} + a_0 M + a_1 C \)

5. Trianglarize \( K^* \):
   \[ K^* = LDL^T \]

C. Step by Step Computation
1. Assume the pseudo-force (force from damper) \( F_{D,x}^i = 0, \dot{u}_i^i = 0 \) solve for \( F_{D,x+\Delta t}^i \) in the first iteration \( i = 1 \) using Eq. (5-8)
2. Calculate the incremental effective load vector from time \( t \) to \( t + \Delta t \):
   \[ \Delta P^* = \Delta P - \Delta F_P + 2C_0 \dot{u}_0 + M[\frac{4}{\Delta t} \dot{u}_0 + 2 \ddot{u}_0] \]
3. Solve for displacement increment from:
   \[ K^* \Delta u = \Delta P^* \]
   and \( \Delta \dot{u} = \frac{2}{\Delta t} \Delta u - 2 \dot{u}_0 \)
4. Update the states of motion at time \( t + \Delta t \):
   \[ u_{t+\Delta t} = u_t + \Delta u; \dot{u}_{t+\Delta t} = \dot{u}_t + \Delta \dot{u} \]
5. Use \( F_{D,x}^{i+1} = 0, \ddot{u}_{t+1}^i = 0 \) and \( \ddot{u}_{t+\Delta t}^{i+1} = \ddot{u}_{t+\Delta t} \) solve for \( F_{D,x+\Delta t}^{i+1} \) using Eq. (5-8).
6. Compute Error:
   \[ |F_{D,x+\Delta t}^{i+1} - F_{D,x+\Delta t}^i| \]
7. If error \( \geq \) tolerance, return to C-1 for further iteration.
8. If error \( \leq \) tolerance, no further iteration is needed. continue to next time step.
response is evaluated in terms of damage to members defined by the ratio of permanent curvature demand versus capacity expressed as (Reinhorn and Valles 1995):

\[
\frac{\phi - \phi'}{\phi_u - \phi'_u} = \frac{\Delta \phi_a}{\Delta \phi_{a_0} \left(1 - \frac{E_h}{4E_{h0}}\right)} = \frac{\Delta \phi_u}{\Delta \phi_{u_0}}
\]

(5-12)

where \(\phi\) indicates the maximum deformation demand, \(\phi'\) indicates the recoverable curvature due to elastic rebound, at maximum curvature, \(\phi_u\) the ultimate curvature capacity and \(\phi'_u\) the elastic rebound at same ultimate curvature, \(\Delta \phi_a\) and \(\Delta \phi_{a_0}\) are the achieved maximum permanent curvature and the ultimate monotonic permanent curvature capacity, respectively. \(E_h\) is the cumulative energy dissipated by the member and \(E_{h0}\) is the energy dissipated monotonically at rupture (ultimate curvature capacity). If \(\Delta \phi_a\) is the maximum permanent curvature in an event, then the index determined by Eq. (5-12) is defined as the "Event Damage Index" (Reinhorn and Valles 1995). If \(\Delta \phi_u\) is the maximum residual curvature, the damage index is defined as the "Residual Damage Index". It should be noted that the ultimate dynamic permanent curvature capacity, \(\Delta \phi_u\), is reduced during an earthquake as a function of the energy dissipation (Reinhorn and Valles 1995). Therefore the damage can be reduced by reducing the hysteretic energy dissipation demand, \(E_h\).

5.2.5 Determining the Monotonic Strength Envelope

An inelastic monotonic envelope defines the force deformation strength of a structure or substructure and can be obtained through a pushover analysis. Static forces
proportional to the story resistance are applied incrementally to the structure and the
deformations are determined along with the internal force distribution. From the structures
Eq. (5-1), neglecting the wind loading \( F_w \):

\[ R(u) = -M(\ddot{u} + \ddot{u}_g I) - C\dot{u} = F_i \] \hspace{1cm} (5-13)

Pre-multiplying both sides by a unit vector, \( I^T = \{1,1,\ldots,1\} \), Eq. (5-13) becomes:

\[ I^T R(u) = -I^T (M\ddot{u}_a + C\dot{u}) = I^T F_i \] \hspace{1cm} (5-14)

where \( \ddot{u}_a \) is the total absolute acceleration, \( \ddot{u} + \ddot{u}_g I \).

The right hand side of the Eq (5-14) is the total base shear, \( BS \):

\[ BS = I^T F_i \] \hspace{1cm} (5-15)

Dividing Eq. (5-13) by (5-14) and using relationship of Eq. (5-15), the inertia forces are
obtained as:

\[ F_i = BS \frac{R(u)}{I^T R(u)} \] \hspace{1cm} (5-16)

The above force distribution is applied incrementally in the pushover analysis by increasing
the base shear:

\[ F_i^k = (BS^{k-1} + \Delta BS^k) \frac{R^{k-1}(u)}{I^T R^{k-1}(u)} \] \hspace{1cm} (5-17)
where \( k \) indicates the step of computation. The distribution of pushover force is based on previous computation step, since data is not available without iteration. The error, 

\[
ERR = BS^k - I^T R^k (u),
\]

involved in the above is minimal. However if the error is substantial, an iteration should be performed using Eq. (5-17) until solution converges.

The deformation is obtained from the incremental analysis:

\[
K_k \Delta u^k = \Delta F_i^k
\]

\[
(5-18)a
\]

in which \( \Delta F_i^k \) can be approximated as:

\[
\Delta F_i^k = F_i^k - F_i^{k-1}
\]

\[
(5-18)b
\]

Solving for \( \Delta u^k \) one can determine the deformation increase. The increase in the internal forces is obtained from:

\[
R^k (u) = K_k \Delta u^k + R^{k-1} (u)
\]

\[
(5-18)c
\]

The stiffness \( K_{k+1} \) for next step is calculated from Eq. (5-3). The procedure determines the resistance envelope at any desired floor, or for the total structure characteristics.

5.2.6 Monotonic Strength Envelope with Braces

The structure stiffness will be enhanced in presence of dampers depending on different stages of the dampers, therefore instead of using the original stiffness of structure, \( K \) from Eq. (5-3), the enhanced stiffness \( K' (=K+\Delta K) \) should be used, since it
includes the contribution of dampers, $\Delta K$. The $\Delta K$ can be evaluated depending the stages of the dampers as:

$$\Delta K = Bk_i$$ \hspace{1cm} \text{(5-19)}

where $k_i$ is the stiffness of the damper $i$ and:

$$B = \begin{bmatrix}
N_j \cos^2 \theta_j & -N_j \cos^2 \theta_j \\
-N_j \cos^2 \theta_j & N_j \cos^2 \theta_j + N_{j-1} \cos^2 \theta_{j-1} & -N_{j-1} \cos^2 \theta_{j-1} \\
& \ddots & \ddots & \ddots \\
& & -N_j \cos^2 \theta_3 & N_j \cos^2 \theta_3 + N_2 \cos^2 \theta_2 & -N_2 \cos^2 \theta_2 \\
& & \ddots & \ddots & \ddots \\
& & & -N_j \cos^2 \theta_1 & N_j \cos^2 \theta_1 + N_1 \cos^2 \theta_1
\end{bmatrix}
$$ \hspace{1cm} \text{(5-20)}

$$k_i = k_{Di}, \quad F_{Di} < F_{Di,\text{break-away}}$$

$$k_i = 0, \quad F_{Di} > F_{Di,\text{break-away}}$$ \hspace{1cm} \text{(5-21)}

where $N_j$ is the number of dampers or unit multiplier for dampers in brace level $j$ with an angle of incidence of $\theta_j$.

The performance of influence of dampers stiffening is evaluated in Sec. 5.3.

5.3 Validation of Structural Model with Friction Dampers

5.3.1 Time History Analysis

The performance of the structure model retrofitted with friction dampers was determined analytically through time history analysis. Bouc-Wen's model, with parameters from Section 2, was used to model the dampers for the test structure presented in Sections 3 and 4 subjected to several simulated earthquakes. The analytical and
experimental displacements and the accelerations of the structure are compared in Fig. 5-3 and 5-4 for El-Centro earthquake and Fig. 5-5 and 5-6 for Taft earthquake. Similar results are obtained for all other earthquakes. The forces in the dampers calculated using Bouc-Wen’s model are shown in Fig. 5-7. The computed maximum forces and displacements in the damper, as well as the total energy dissipated are in good agreement with the experimental results.

5.3.2 Monotonic Pushover Analysis

The validity of pushover analysis was verified also with experimental data. The analysis was performed according to the procedure obtained in Sec. 5.2. Fig. 5-8 indicated the variation of total structure resistance in terms of base shear (foundation reaction, Eq. 5-14 and 5-15) as a function of the displacement at the top of the structure. The stiffening effect at various stages of structural deformation is presented in Fig. 5-8. The initial strength resistance including the dampers (Eq. 5-25, i.e. \( K + \Delta K \) in Fig. 5-8) can be up to 2.5 times larger than the original, and the final strength resistance (assume all dampers were at stage of slip) is equal to the resistance of the original structure plus a constant resistance from dampers.

Overall the pushover analysis is representative to the variation of total internal forces in structure due to the dynamic response. The introduction of dampers only increase the initial stiffness of the structure and once the dampers are slipping, the structure only increases certain strength without any stiffening. The increase of force demands in structure joints and foundation is limited. (see also Section 6).
Figure 5-3 Comparison of Experimental and Analytical Displacement for El-Centro 0.3g (with Six Tekton Friction Dampers)
Figure 5-4  Comparison of Experimental and Analytical Acceleration for El-Centro 0.3g (with Six Tekton Friction Dampers)
Figure 5-5 Comparison of Experimental and Analytical Displacement for Taft 0.2g (with Six Tekton Friction Dampers)
Figure 5-6  Comparison of Experimental and Analytical Acceleration for Taft 0.2g (with Six Tekton Friction Dampers)
Figure 5-7  Comparison of Damper Forces for El-Centro Earthquake PGA 0.3g (with Six Tekton Friction Dampers)
Figure 5-8 Structural Resistance in Presence of Friction Dampers
SECTION 6

SIMPLIFIED EVALUATION OF INELASTIC RESPONSE WITH SUPPLEMENTAL DAMPING

6.1 Response Spectra for Elastic Systems

The representation of structural response of elastic structures becomes more relevant using spectral approach monitoring simultaneously the acceleration (force) and displacement responses. The spectral representation of peak inertia forces versus the peak displacement response was suggested for evaluation of elastic structures (Kircher 1993a) and for inelastic structures (Freeman 1993, Kircher 1993b).

6.1.1 Composite Response Spectra for Single Degree of Freedom (SDOF)

The acceleration response spectrum indicating the maximum acceleration, $S_a(T, \xi)$, is dependent on the period, $T$, and the damping of the SDOF oscillator, $\xi$. The maximum inertia force (or base shear, BS), is obtained:

$$BS = \left( \frac{W}{g} \right) S_a(T, \xi) \tag{6-1a}$$

or

$$BS/W = \frac{S_a(T, \xi)}{g} \tag{6-1b}$$
The displacement response spectrum can be obtained by direct computation, \( S_d(T, \xi) \), or by transformation of acceleration spectra into a pseudo displacement spectrum:

\[
PS_d(T, \xi) = \frac{S_a(T, \xi)}{(2\pi \xi T)^2}
\] .................................................................(6-2)

The plot of base shear spectra versus displacement response spectra are shown in Fig. 6-1 as composite response spectra. A line passing through origin with a slope of \((2\pi/T)^2\) will intersect the spectral line for \( \xi_i \) at a point with coordinates indicating the response spectra of acceleration \( S_a(T, \xi_i) \) and of displacement \( PS_d(T, \xi_i) \). If \( S_a(T, \xi_i) \) is used rather than \( PS_d(T, \xi_i) \), then the line with slope of \((2\xi/T)^2\) will indicate only approximately the displacement.

6.1.2 Composite Spectra for Multi-Degree of Freedom (MDOF)

The acceleration response of any degree of freedom \( i \) due to a given spectral acceleration is:

\[
\ddot{u}_k = \left\{ \sum_j [\Phi_{kj} \Gamma_j S_a(T_j, \xi_j)]^2 \right\}^{1/2}
\] .................................................................(6-3)

in which \( \Phi_{kj} \) is the modal shape \( j \) (mass normalized i.e. \( \sum m_k \Phi_{kj}^2 = 1 \) and \( \Gamma_j \) is the modal participation factor \( (= \sum m_k \Phi_{kj}^2) \).

\[
u_k = \left\{ \sum_j [\Phi_{kj} \Gamma_j S_a(T_j, \xi_j)]^2 \right\}^{1/2}
\] .................................................................(6-4)
Figure 6-1 Composite Response Spectra for SDOF
The above definitions are based on SRSS superposition.

6.1.2.1 Composite Spectra for a Single Mode

For a single mode contribution, the modal component of acceleration and displacement, can be expressed for a single mode i setting j=1 in Eq. (6-3) and (6-4). Varying the period, $T_i$ from $T_1$ to $T_2$ range (selected for the description of the spectrum), then the composite spectral modal response can be defined as:

$$S_{a_i}^{(i)}(T, \xi) = \Phi_i \Gamma_i S_a(T, \xi); \quad T_i \leq T \leq T_2 \hspace{1cm} \text{Eq. (6-5a)}$$

$$S_{a_i}^{(i)}(T, \xi) = \Phi_i \Gamma_i S_a(T, \xi); \quad T_i \leq T \leq T_2 \hspace{1cm} \text{Eq. (6-5b)}$$

The composite spectra is defined as a function of $S_{a_i}/g$ vs $S_{a_i}$ defined above, similarly with the spectra for SDOF (Fig. 6-1). The modal base shear is obtained from Eq. (6-5b)

$$BS/W = \Gamma_i S_a(T, \xi)/g \hspace{1cm} \text{Eq. (6-6)}$$

The composite spectra can be defined for the maximum base shear versus the maximum displacement response at any degree of freedom, $k$, by adjusting the index in Eq. (6-5). Charts similar to Fig. 6-1 can be developed for single mode.

6.1.2.2 Composite Spectra Including Higher Modes

The response in Eq. (6-3) can be written as:
\[ \ddot{u}_k = \left\{ \sum_j \left[ \Phi_{kj} \Gamma_j S_a \left( T_0 \frac{T_j}{T_0}; \xi_0 \frac{\xi_j}{\xi_0} \right) \right] \right\}^{\frac{1}{2}} \]  
(6-7)

\[ u_k = \left\{ \sum_j \left[ \Phi_{kj} \Gamma_j S_d \left( T_0 \frac{T_j}{T_0}; \xi_0 \frac{\xi_j}{\xi_0} \right) \right] \right\}^{\frac{1}{2}} \]  
(6-8)

in which the period \( T_j \) was expressed as a ratio \( (T_j/T_0) \) times \( T_0 \), the fundamental period, similarly the damping ratio \( \xi_j \). Assuming that \( (T_j/T_0) \) is constant for any mode in respect to the first, independently of the value of \( T_0 \), it is possible to define a maximum peak for \( \ddot{u}_k \) and \( u_k \) including the higher modes, by varying \( T_0 \) between two limits, \( T_1 \) and \( T_2 \), defining as the spectral range. The composite spectrum, can be defined therefore by:

\[ S_{u_k}(T, \xi) = \left\{ \sum_j \left[ \Phi_{kj} \Gamma_j S_a \left( T, \xi, \frac{T_j}{T}, \frac{\xi_j}{\xi} \right) \right]^{\frac{1}{2}} \right\} \]  
(6-9a)

\[ S_{s_k}(T, \xi) = \left\{ \sum_j \left[ \Phi_{kj} \Gamma_j S_d \left( T, \xi, \frac{T_j}{T}, \frac{\xi_j}{\xi} \right) \right]^{\frac{1}{2}} \right\} \]  
(6-9b)

and plotted as the chart in Fig. 6-1.

Any other important response quantities can be derived from the definitions in Eq. (6-9). For example the base shear, \( BS \) can be determined:

\[ BS/W = \left\{ \sum_j \left[ \Gamma_j^2 S_d \left( T, \xi, \frac{T_j}{T}, \frac{\xi_j}{\xi_0} \right) \right]^{\frac{1}{2}} \right\} \]  
(6-10)
Using the expression in Eq. (6-10) and (6-9), one can develop a composite spectrum similar to Fig. 6-1 for SDOF.

Fig. 6-2 presents the composite spectra for the structural model studied in Section 3. The composite spectra based on single mode contribution (Eq. (6-6) and (6-5)b) is shown in Fig. 6-2a. The composite spectra based on three modes (Eq. (6-10) and Eq. (6-9)) is shown in Fig. 6-2(b) for comparison. Differences can be noted at high periods, however, for most purposes, the differences are minor otherwise.

6.2 Evaluation of Seismic Demand in Elastic Structures

6.2.1 Response without Supplemental Damping

The equation of motion of an elastic system is defined as:

\[ M\ddot{u} + C\dot{u} + Ku = -M\ddot{u}_g \] .............................................................................. (6-11)

or grouping the terms:

\[ M(\ddot{u} + \ddot{u}_g) + C\dot{u} = -Ku \] .............................................................................. (6-12)

The extreme response requires that:

\[ \left[M(\ddot{u} + \ddot{u}_g) + C\dot{u}\right]_{\text{max}} = -Ku_{\text{max}} \] .............................................................................. (6-13)a

If damping is indicated in the first term, (as shown in Eq. (6-13)), then this term indicates the inertia forces influenced by structure damping, i.e.
Figure 6-2 Composite Response Spectra for MDOF
(a) Single Mode Contribution, (b) Three Mode Contribution
\[ \left[ M\left( \ddot{u} + \dddot{u} \right) + C\dot{u} \right]_{\max} = -MS_a \left( T_0, \xi_0 \right) \] ..............................................(6-14)

The right side of Eq. (6-13) indicates:

\[ Ku_{\max} = KS_a \left( T_0, \xi_0 \right) \] .............................................................(6-15)

in which \( T_0 \) indicates the fundamental period.

Eq. (6-13) can be rewritten as:

\[ MS_a \left( T_0, \xi_0 \right) = KS_a \left( T_0, \xi_0 \right) \] .............................................(6-16)

Using the composite spectrum, Eq. (6-16) shows that the ratio of \( S_a/S_a = \left( \frac{2\pi}{T_0} \right)^2 \) is a line which intersects at the response quantities (see Fig. 6-1).

Therefore, to determine the actual response using the composite spectrum, an intersection of the spectral curve with the structure stiffness/mass properties line with the slope \( \tan \alpha = K/M = \left( \frac{2\pi}{T_0} \right)^2 \) is required. The intersection point indicate the structural response in base shear and displacement terms (see point A in Fig. 6-3).

6.2.2 Response with Supplemental Damping

The friction damper force can be represented by Eq. (2-1) as:

\[ F_D = k_D U, \quad for \ |F_D| \leq \mu_{\text{break-away}} N \] ..........................................................(2-1)a, repeat

\[ F_D = \mu_{\text{min}} N, \quad for \ \mu_{\text{min}} N \leq |F_D| < \mu_{\text{break-away}} N, \text{ after sliding occurred.} \] (2-1)b, repeat
Figure 6-3  Response-Demand Using Composite Spectra
when added to Eq. (6-12), Eq. (6-13)a becomes:

\[
\begin{bmatrix}
    M \left( \ddot{u} + \ddot{u}_g \right) + Cu \\
\end{bmatrix}
\]

\[\Rightarrow -(K + \Delta K)u_{\text{max}}, \quad \text{for } |F_D| \leq \mu_{\text{break-away}}N \] ..............(6-13)b

\[
\begin{bmatrix}
    M \left( \ddot{u} + \ddot{u}_g \right) + Cu \\
\end{bmatrix}
\]

\[\Rightarrow -(N\mu_{\text{min}} + \Delta K u_{\text{max}}), \quad \text{for } \mu_{\text{min}}N \leq |F_D| \leq \mu_{\text{break-away}}N \] (6-13)b

which indicates a change of initial slope in the stiffness/mass line in Fig. 6-3 to (K+\Delta K)/W (and then with constant strength increase after slip C point) and a shift in the original spectral line from \(\xi_0\) to \(\xi_0 + \Delta \xi\) characteristics to the increase from C to C + \(\Delta C\).

It can be noted that the stiffening alone (K to K+\Delta K) has the tendency to reduce deformations but increase the force (base shear) demand (point B) in Fig. 6-3. The increase in damping along with stiffness increase (C to C+\Delta C) reduces both deformation and force demand (point C in Fig. 6-3).

### 6.3 Evaluation of Motion of Inelastic Structures

The equation of motion of an inelastic system (Eq. 5-1):

\[
M \ddot{u} + C \dot{u} + R(u) = -M \ddot{u}_g \] .................................................................(5-1)Repeat

in which \(R(u)\) is the structure strength determined according to the procedure in section 5.2.5. Similarly with Eq. (6-13), the maximum response can be determined from:

\[
\begin{bmatrix}
    M \left( \ddot{u} + \ddot{u}_g \right) + Cu \\
\end{bmatrix}
\]

\[= MS_g(T, \xi) = R(u)_{\text{max}} \] .................................................................(6-17)
Eq. (6-17) suggests that the maximum deformation is obtained at the intersection of the structure resistance $R(u)$ with the acceleration spectral lines as shown in Fig. 6-4a. The spectral lines based on NEHRP, 1994 are used in Fig. 6-4 for an MDOF composite spectrum (see Section 6.1.2.2), for the test structure in Section 4. If the structure was elastic, the base shear would have been larger, while for the inelastic structure, the base shear response is smaller but accompanies by larger deformation.

6.3.1 Response Neglecting Hysteretic Damping

The structure dissipates energy during inelastic excursions (Bracci et al. 1992). Neglecting this energy, the damping in inelastic response will remain as the original, as shown in Fig. 6-4. However, neglecting the hysteretic damping, displacements and base shear larger than expected are produced if the response spectrum is a monotonically changing function.

6.3.2 Response Considering the Hysteretic Damping

The hysteretic energy dissipation can be interpreted as an increase in the "viscous" damping. In such case the response is obtained at the intersection of the elastic strength function $R(u)$ with the composite spectral lines for an increased damping ratio $\xi_2 = \xi_1 + \Delta\xi$. An example of such response is shown in Fig. 6-5. The equivalent damping increase was measured from experiments using the equivalent frequency response for the structure subjected to three intensities ground shaking, i.e. Taft acceleration with PGA of 0.05g, 0.20g and 0.30g. The intersections of the composite spectra and the strength capacity.
Figure 6-4  Demand in Inelastic Structure Using Composite Spectra
Figure 6-5  Composite Spectra vs Capacity of Structure for Taft 0.05g, 0.20g and 0.30g for 2% and 10% Critical Damping. Tested Damping Ratios 4.6%, 8.2% and 3% for above Motions, Respectively.
function, $R(u)$ are very close to the experimental points. This indicates that the approach can determine the response of forces and displacements with an acceptable approximation.

### 6.3.2.1 Estimate of Equivalent Hysteretic Damping

For practical purpose however, the calculation of the equivalent damping is a complicated issue. The "viscous" equivalent damping depends on the hysteretic energy dissipation per cycle (Fig. 6-6):

$$E_{he} = 4\gamma P_y (u_{\text{max}} - u_y)$$  \hspace{1cm} (6-18)

in which $g$ is the ratio of the area enclosed in the hysteresis versus the area of the parallelogram $[4P_y (u_{\text{max}} - u_y)]$. This factor is influenced by bond slip or "pinching" in reinforced concrete elements ($\gamma_c = 0.4 - 0.6$) or by the Baussinger effects in steel structures [$\gamma_s = 0.6 - 0.9$]. The equivalent damping ratio is defined as:

$$\Delta \zeta_{eq} = \frac{E_{he}}{4\pi E_{pv}}$$  \hspace{1cm} (6-19)

with the notation shown in Fig. 6-6. The equivalent increase in the damping ratio is therefore obtained as:

$$\Delta \zeta_{eq} = \frac{2\gamma \mu(\mu - 1)}{\pi \mu [1 + \alpha(\mu - 1)]}$$  \hspace{1cm} (6-20)

in which $\mu$ is the ductility defined as $\mu = u_{\text{max}} / u_y$. It is evident that the damping increase is a function of amplitude (ductility) per cycle. Earthquake response is neither cyclic nor
Figure 6-6  Cyclic Hysteretic Energy Dissipation
constant amplitude. Therefore the increase in damping can be determined only by approximations from response characteristics.

Using a linear model for which the maximum ductility is replaced by a rms., $\sigma_{\mu}$, in Eq. (6-20) instead of $\mu_s$, the equivalent damping is obtained as:

$$\Delta\xi_{eq} = \frac{2\gamma(\sigma_{\mu} - 1)}{\pi\sigma_{\mu}[1 + \alpha(\sigma_{\mu} - 1)]} \quad (6-21)$$

Assuming a probability density function such as a Gaussian distribution with a zero mean, the relation between the rms. (standard deviation) and the peak (assuming a probability of occurrence of 97.7%) is:

$$\mu_{max} = 2\sigma_{\mu} \quad (6-22)$$

Therefore the equivalent damping can be approximated from Eq. (6-21) with Eq. (6-22):

$$\Delta\xi_{eq} = \frac{4\gamma(\mu_{max} - 2)}{\pi\mu_{max}[2 + \alpha(\mu_{max} - 2)]} \quad (6-23)$$

which produces acceptable agreement for maximum deformation ductilities larger than 2. For smaller values the damping increase is negligible and should not be considered. Table 6-1 shows the damping increase for an reinforced concrete structure ($\gamma=0.5$) for various maximum ductilities. The damping obtained as shown above can estimate grossly the increase in damping in the test structure due to the hysteretic behavior. Further investigations might be necessary for improved results.
Table 6-1 Increase in effective damping ratio, Δ\(\xi_{eq}\) (for \(\gamma=0.5\))

<table>
<thead>
<tr>
<th>(\mu_{\text{max}})</th>
<th>2.00</th>
<th>2.05</th>
<th>2.10</th>
<th>2.20</th>
<th>2.50</th>
<th>3.00</th>
<th>3.50</th>
<th>4.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\alpha=0.02)</td>
<td>0%</td>
<td>1%</td>
<td>2%</td>
<td>3%</td>
<td>6%</td>
<td>11%</td>
<td>13%</td>
<td>16%</td>
</tr>
<tr>
<td>(\alpha=0.10)</td>
<td>0%</td>
<td>1%</td>
<td>2%</td>
<td>3%</td>
<td>6%</td>
<td>10%</td>
<td>13%</td>
<td>14%</td>
</tr>
</tbody>
</table>

6.4 Evaluation of Response of Inelastic Structure with Supplemental Damping

The suggested evaluation uses the composite spectrum approach outlined above. The response is obtained at the intersection of the composite spectrum lines with the inelastic resistance line obtained from push-over analysis, including the influence of supplemental dampers as presented in Section 5.3.2. The influence of stiffening and damping is evaluated below.

6.4.1 Influence of Damping Increase

If the damping devices have only damping characteristics, neglecting the initial stiffness increase, the structure resistance (capacity) remains as before retrofit (see Fig. 5-11, without dampers and Fig. 6-7). If the response without supplemental dampers is represented by point A (\(\xi = 10\%\)) in Fig. 6-7, an increase in damping will shift the response to point B (\(\xi = 20\%\)). The displacement response is reduced primarily with some reduction of base shear. However, the initial stiffness and strength increase is inevitable for friction devices. The capacity considering initial stiffness increase and strength increase has to be considered as follows.
6.4.2 Influence of Stiffening due to Supplemental Dampers

As previously outlined in Section 5.3.2, the dampers have a substantial contribution to stiffening at initial stage (see also Fig. 6-7). The influence of stiffening can be seen in the shift of point B to D in Fig. 6-7. The influence of stiffening and strengthening contributes to a further reduction of displacement response and increase in the base shear demand (although minor). A substantial stiffening and strengthening will increase the base shear demand substantially.

6.4.3 Influence of Dynamic Strength

It should be noted that the influence of supplemental damping in inelastic structures is to decrease the deformation of the structure and influence slightly the base shear demand, in many instance by a minor increase. However, it should be noted that the total shear includes the influence of the original structural elements, for which the capacity is indicated by the original line (point E in Fig. 6-7) at the maximum deformation response, and the influence of the dampers for which the forces are the difference between points D and E in same figure. Fig. 6-7 shows therefore that the forces in the original structural elements are reduced even in presence of stiffening. Moreover, the reduction in the deformation is also accompanied by a reduction of the demand for hysteretic energy dissipation which presents deterioration and extensive damage in structural elements (see also Section 4).

The minor increase in the base shear or in many cases the minor increase in the story shear forces may prove to be critical in the design of the load transfer path (i.e.
GROUND MOTION SPECTRA ACCORDING TO NEHRP 94, SOIL TYPE 1

RESPONSE WITH STIFFENING (ESTIMATED)
RESPONSE WITHOUT STIFFENING (ESTIMATED)
CAPACITY WITH DAMPERS
CAPACITY WITHOUT DAMPERS
EARTHQUAKE TESTING (WITH DAMPERS)
EARTHQUAKE TESTING (WITHOUT DAMPERS)

Figure 6-7 Influence of Supplemental Damping

DISPLACEMENT AT TOP OF BUILDING (% OF BUILDING HEIGHT)
BASE SHEAR (% OF BUILDING WEIGHT)
connections, joints, foundations, etc.). Therefore, for design purposes, the maximum
deformation demand can be determined conservatively including no stiffening, while the
force demand can be determined conservatively from the capacity curve including
stiffening and strengthening. The experimental study for the test structure shows this trend
(see Fig. 6-8 and 6-9). The composite spectra was calculated using MDOF calculations
(Section 6.1.2.2) while the response of the original structure is found on the original
capacity curve, the response of the retrofitted structure with supplemental dampers fits the
prediction from capacity curve with initial stiffening and strengthening after certain
deformation (see Fig. 6-8 and 6-9), as already indicated in the discussion in Section 5.3.2.

The original structure (retrofitted by jacketing and damaged by prior tests) showed
an "inherent" damping of 3% to 5% in mild inelastic response (ductilities below 2). The
damping increase in the structure was entirely due to damping devices.

Although the composite spectrum diagram indicates adequately trends in the
structural response, a better estimate of the damping characteristics, or a better estimate of
the composite spectrum, is required in order to obtain a reliable estimation tool. The
damping estimated through frequency analysis and through equivalent analytical tools (see
Table 4-4) do not fit perfectly the damping increase showed in the composite spectra in
Fig. 6-8 and 6-9. The experimental results show smaller "equivalent" damping than
estimated by other means.

The composite spectrum is using information from the elastic response, while the
structural response is inelastic. In the range of the experiment, the inelastic displacement
Figure 6-8  Evaluation of Structural Response for El-Centro Earthquake, PGA 0.3g
Evaluation of Structural Response for Taft Earthquake, PGA 0.2g
It should be noted however, that using the spectral curves (developed according to NEHRP, 1994) instead of the individual motions used during testing, the estimate using approximated damping calculations (based on Table 4.3 column (5)) leads to results close to those from experiments (see Fig. 6-10 and 6-11). The spectral curves represent an average of multiple motions and the estimates are not affected by the response spectrum fluctuation when minor error in the estimate of structural parameters are present.

6.5 Evaluation of Experimental Response (Summary)

The experimental response of test structure was evaluated for the retrofit using friction dampers and the results are summarized in Table 4-4a and 4-4b and in Fig. 6-12 and Fig. 6-13 for the structure tested with and without dampers. The results for the other motions cannot be compared with the response without dampers since the unretrofitted structure could not be tested with such motions without the risk of complete collapse. The major findings from the comparison and the evaluation in view of the simplified composite-spectrum approach are presented below:

(a) The response related to displacements or drifts shows substantial reductions, from 30% to 60%, at all stories of the structure. This can be easily derived from the simplified composite spectrum approach presented in the previous section. The response
GROUND MOTION SPECTRA ACCORDING TO NEHRP 94, SOIL TYPE 1

ANALYTICAL SPECTRUM ESTIMATE (WITH)
ANALYTICAL SPECTRUM ESTIMATE (W/O)
CAPACITY OF THE BUILDING (WITH)
CAPACITY OF THE BUILDING (W/O)
EARTHQUAKE TESTING (WITH)
EARTHQUAKE TESTING (WITHOUT)

Figure 6-10 Evaluation of Response Using NEHRP Spectra (PGA=0.3g)
Figure 6-11  Evaluation of Response Using NEHRP Spectra (PGA=0.2g)
Figure 6-12  Summary of Experimental Response of Tested Structure Model  
(El-Centro, PGA 0.3g)
Figure 6-13  Summary of Experimental Response of Tested Structure Model (Taft, PGA 0.2g)
moves back on the capacity curve (see Fig. 6-7 which is flat in the inelastic range) to the increased damping spectra line, reducing substantially the displacements.

(b) The response related to accelerations (Fig. 6-12(c)), overturning moments (Fig. 6-12(d)), story forces (Fig. 6-12(f)) or story shear coefficients (Fig. 6-12(i)) show very little change, some reduced and some increased. The composite spectrum approach indicates this fact following the flat portion of the capacity diagram which has a small slope, on one hand, and is following stiffening patterns, on the other hand. The forces where increased limited amount since the friction dampers have only initial stiffness increase as shown in previous sections. The expected forces and accelerations can be derived from the composite spectrum provided good evaluation of expected damping is possible.

(c) The internal shear force (measured during the experiments) in the columns of the structure retrofitted with friction dampers are smaller than the forces in the unretrofitted structure, by a small amount (Fig. 6-12(f)). Although the total shear force is reduced insignificantly, the forces in the column alone are reduced more substantially 20% - 50%. This reduction is expected in view of the composite spectra and capacity curves as explained in Section 6.4.3 by Fig. 6-7, points A, B, C, D and E). The reduction of the shear forces in the columns depends primarily on the inelastic state at maximum response. If large inelastic excursions are expected, then the reduction in forces might be smaller than if smaller inelastic excursions occur, depending on the "flatness" of post-yielding
capacity curve (compare reductions of 2nd story shears in structure, Fig. 6-12(f) and 6-13(f)).

(d) The forces in the friction dampers reach their maximum before the forces in columns do, and then keep a constant value for larger deformation. The connections and columns should be designed for combination of maximum forces and friction damper slip force and so does the foundation.

(e) A summary of testing results of the retrofitted structure with various damping devices (as indicated in the overall research program description in Section 1) is presented in Fig. 6-14 and 6-15. Fluid viscous devices, viscoelastic devices and special viscous walls were sized to fit a desired retrofit scheme. Although the designs were similar, due to construction constrains the resulting devices were different in damping capacity and stiffening characteristics, such that their influence can not be directly compared.

However, the trends of their influence on the structure can be evaluated and quantified using the capacity and composite spectrum approach. The influence of all devices is to reduce deformations and drifts (Fig. 6-10(a), (g)), while increasing or minimally reducing the overall structural forces (Fig. 6-14(d)(f)). The viscous devices (the subject of this report) have a minimal influence on the story forces among the other devices since their stiffening effect is minimal. The viscoelastic braces tested in the same structure have similar damping, but slightly higher stiffness that contributes to an overall increase of story forces.
Figure 6-14 Summary of Experimental Response of Tested Structure Model with Various Dampers (El-Centro, PGA 0.3g)
Figure 6-15  Summary of Experimental Response of Tested Structure Model
with Various Dampers (Taft, PGA 0.2g)
The above trends validate the evaluation using the capacity and composite spectrum approach. Using this tool, it is possible to size damping devices and the structural components to achieve the desired goal of the retrofit, which is reduction of deformations and hysteretic energy dissipation demands that lead to damage. However, a complete nonlinear analysis is further necessary for the qualification of the final design.
SECTION 7

CONCLUSIONS

A combined experimental and analytical study of reinforced concrete structures retrofitted with friction dampers is presented herein. Shaking table tests of a 1:3 scale R/C frame structure with friction damping braces installed in the mid-bay of the frame with different configurations were conducted. A comprehensive component test program was also conducted on the friction dampers over a frequency range between essentially 1 Hz and 4 Hz. The inelastic behavior of the structure retrofitted using friction dampers incorporated in braces was investigated. The analytical modeling of friction damping devices was presented and models were implemented in IDARC2D, ver. 4.0 a platform for inelastic analysis for reinforced concrete structure with damping devices.

The important observations and conclusions of this study are summarized below:

(1) The retrofit of damaged R/C structure with friction damping braces produces satisfactory response during earthquakes. The damping enhancement contributes to the reduction of maximum deformations, primarily, and modifies only slightly the structural forces transmitted to the foundations.

(2) The dampers show stiffening characteristics at initial stage and show only strengthening afterwards. Stiffening and strengthening effects are almost not affected by frequency.
(3) The period of the structure varies with the intensity of the earthquake which will shift the structural frequency away from resonant frequency.

(4) Stiffening of structure from the damping devices leads to reduction of system's deformations. However, it may cause minor accelerations' increase (or total large shear increase). Strengthening of structure from damping devices increases the capacity of the structure.

(5) Although, total base shear could be increased somewhat, the internal shear forces in the original system retrofitted (i.e. columns, beams, etc.) are always reduced. The total structure shear includes the increased forces in dampers, synchronous with the forces in members, therefore subtracting this influence results in smaller forces in the original system. Therefore, the "structure's retrofit with dampers benefits in lowering the internal shear forces, although not in the same measure as the reduction of its deformations.

(6) The hysteretic behavior of dampers provides the main contribution to forces reduction of the structural response. However, the forces in dampers may transfer to columns so as to increase the axial force in the columns. A structural analysis should be made to determine the transfer load path.

(7) The corrosion problem of friction interfaces should be considered for long time period usage. The composition of the interfaces is of paramount importance for ensuring the longevity of the device.
(8) The dampers can be modeled by Bouc-Wen’s model which is a smooth bilinear hysteretic model.

(9) The transfer load path and the influence of stiffening and strengthening of dampers can be obtained from a monotonic inelastic "push-over" analysis of structure as suggested herein. The dampers contribute their stiffening and strengthening properties to the increase in the overall capacity of structure. At large deformations the contribution comes from the strengthening property of the damper. At smaller deformation the contribution comes from stiffening property of the damper.

(10) The primary effect of dampers is the reduction of demand for hysteretic energy dissipation by the gravity load carrying structural members. Such a reduction that may be up to 80%-90%, leads to a substantial reduction of structural damage in the members due to low cycle fatigue (as reflected by the damage analysis) presented herein.

(11) Composite spectrum, acceleration/force versus deformation spectra combined with elastic analysis, can provide a good estimate of the peak structural response if interested with the "push-over" capacity curve. Although the accuracy of such estimate depends on the ability to determine the damping equivalent of inelastic (hysteretic) energy dissipation, The peak demands and the trends in the retrofit applications obtained from such approach can assist the design engineer in determining the initial design values. A more extensive nonlinear analysis is their required for final qualifications of design.
(12) The dampers size and position can also be determined using simple optimal structural control approach as presented by Gluck et al., 1995.

(13) Although the trends are similar for retrofit using other types of dampers, i.e. viscoelastic, fluid, etc., their modeling and general behavior has particular characteristic as shown in the other reports.

(14) Finally, the retrofit using these dampers may require minimal interference with the existing structural system and only minor enhancements of reinforcement in connections or local jacketing might be necessary.
SECTION 8
REFERENCES


8-2


APPENDIX A

A 1-1 Reinforcement Details

The following provides details of the reinforcing steel used in the model based on scale factor of 3 for geometric length similitude. Detailed information is presented by Bracci et al., (1992a), but is repeated here for sake of completion of this report.

The slab steel in the prototype structure was designed by the direct design method of the ACI 318/83. The design required #3 rebars at 6 in. spacing in different sections of the slab. To avoid excess labor in the construction of the 3-story model, a 2 in. square mesh composed of gauge 12 galvanized wires is chosen for acceptable similitudes of strength and geometric spacing length. Since the slab strength is not the main emphasis for this study, the slight disparities of slab steel placement due to the mesh are considered satisfactory for the experiment. Figure A-1 shows the layout details for the top and bottom reinforcing steel mesh in the slab. The longitudinal (direction of motion) and transverse (perpendicular to the direction of motion) beam reinforcement details for the model are shown in Fig. A-2. Figure A-3 shows the reinforcement details for the columns in the model based on the prototype design.

A 1.2 Model Materials

The following outlines the materials used in the construction of the model. It is to be noted that the materials used in the model are identical to materials in assumed prototype structure (Bracci et al., 1992a). Therefore the scale factors were appropriately developed based on the principles of modeling the same acceleration and material.

A 1.2.1 Concrete properties

The concrete mix analysis and design was based on trial mixes from various recipes and a design mix was established for a 28 day target strength of 3500 psi, slump of 4 in., and maximum aggregate size of 1/2 in (#1 crushed stone). Table A-1 shows the mix formula for a one cubic yard batch of concrete.

The mix formulation is based on a saturated, surface dry concrete sand. The water : cement (: sand : stone ) ratio is 0.5 : 1.0 (: 3.0 : 3.6). The full gradation analysis of the aggregates in the concrete mix is shown in Fig. A-4.
FIGURE A-1 Layout of Slab Steel Reinforcement
Longitudinal Beams (North-South)

Transverse Beams (East-West)

FIGURE A-2a Details of the Beam Steel Reinforcement
Beam Sections

FIGURE A-2b Details of the Beam Steel Reinforcement (Continued)
(a) Exterior Section

(b) Interior Section

(c) Section Y-Y

FIGURE A-3 Details of the Column Steel Reinforcement
FIGURE A-4 Gradation Analysis of the Concrete Mix

FIGURE A-5 Average Concrete Specimen Strength Versus Time
Table A-1 Mix Design Formula for the Model Concrete

<table>
<thead>
<tr>
<th>Ingredient</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type I Cement</td>
<td>490 lb</td>
</tr>
<tr>
<td>Concrete Sand</td>
<td>1487 lb</td>
</tr>
<tr>
<td>#1 Crushed Stone</td>
<td>1785 lb</td>
</tr>
<tr>
<td>Water</td>
<td>242 lb</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>39.2 oz</td>
</tr>
<tr>
<td>Micro-Air</td>
<td>2.9 oz</td>
</tr>
</tbody>
</table>

A substantial variation can be observed in the mix strengths for the different components, even though all mixes had the same target strength (see Table A-2). The final strengths were very sensitive to moisture variations in the materials and the widely varying ambient temperatures at the time of construction. The variation of strength versus time is shown in Fig. 3-5, which indicates asymptotic stabilization of concrete strength.

Table A-2 Concrete Properties of the Model Structure

<table>
<thead>
<tr>
<th>Pour Number and Location</th>
<th>( f'_{c} ) (ksi)</th>
<th>( E_{c} ) (ksi)</th>
<th>( \varepsilon_{cc} ) (strains)</th>
<th>( \varepsilon_{spall} ) (strains)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Lower 1st Story Columns</td>
<td>3.38</td>
<td>2920</td>
<td>0.0020</td>
<td>0.011</td>
</tr>
<tr>
<td>2. Upper 2nd Story Columns</td>
<td>4.34</td>
<td>3900</td>
<td>0.0020</td>
<td>0.017</td>
</tr>
<tr>
<td>3. 1st Story Columns</td>
<td>4.96</td>
<td>3900</td>
<td>0.0021</td>
<td>0.009</td>
</tr>
<tr>
<td>4. Lower 2nd Story Column</td>
<td>4.36</td>
<td>3900</td>
<td>0.0026</td>
<td>0.014</td>
</tr>
<tr>
<td>5. Upper 2nd Story Column</td>
<td>3.82</td>
<td>3360</td>
<td>0.0022</td>
<td>0.020</td>
</tr>
<tr>
<td>6. 2nd Story Slab</td>
<td>2.92</td>
<td>2930</td>
<td>0.0015</td>
<td>0.020</td>
</tr>
<tr>
<td>7. 3rd Story Columns</td>
<td>3.37</td>
<td>3800</td>
<td>0.0019</td>
<td>0.020</td>
</tr>
<tr>
<td>8. 3rd Story Slab</td>
<td>4.03</td>
<td>3370</td>
<td>0.0021</td>
<td>0.012</td>
</tr>
</tbody>
</table>
The reinforcing steel uses a mix of #11 & #12 gage wires and D4, D5 annealed deformed bars. The summary of their properties is given in Table A-3.

Table A-3 Reinforcing Steel Properties of the Model Structure

<table>
<thead>
<tr>
<th>Bar</th>
<th>$d_b$ (in)</th>
<th>$A_b$ (in$^2$)</th>
<th>$f_y$ (ksi)</th>
<th>$E_s$ (ksi)</th>
<th>$f_{max}$ (ksi)</th>
<th>$\varepsilon_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>#12 ga.</td>
<td>0.109</td>
<td>0.0093</td>
<td>58</td>
<td>29900</td>
<td>64</td>
<td>0.13</td>
</tr>
<tr>
<td></td>
<td>0.120</td>
<td>0.0113</td>
<td>56</td>
<td>29800</td>
<td>70</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>0.225</td>
<td>0.0400</td>
<td>68</td>
<td>31050</td>
<td>73</td>
<td>0.15</td>
</tr>
<tr>
<td></td>
<td>0.252</td>
<td>0.0500</td>
<td>38</td>
<td>31050</td>
<td>54</td>
<td>-</td>
</tr>
</tbody>
</table>

The D4 rebar was also annealed at different temperatures between 900°F and 1140°F to produce a yield strength between 49 and 73 ksi for yield force similitude with a #6 rebar. At a temperature of 1140°F, the average yield strength consistently reached was 68 ksi. Based on yield force similitude, the D4 rebar represented a #6 rebar with a yield strength of 55.6 ksi. Since a grade 40 steel has yield strengths between 40 and 60 ksi, the D4 rebar satisfied similitude with a #6 rebar. Both the original and annealed stress-strain relationships for the D4 and D5 rebars are shown in Fig. A-6.

![Stress-Strain Graph](image)

FIGURE A-6 Measured Representative Stress-Strain Relationships of the Reinforcing Steel
## APPENDIX A-3

### SCALING FACTORS FOR MODELING OF DYNAMIC BEHAVIOR

<table>
<thead>
<tr>
<th>Quantity</th>
<th>General Case</th>
<th>Same Material and Acceleration (Model)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Required</td>
<td>Provided</td>
</tr>
<tr>
<td>Geometric Length, l</td>
<td>(\lambda_l = ?)</td>
<td>(\lambda_l = 3.00) (\lambda_l = 3.00)</td>
</tr>
<tr>
<td>Elastic Modulus, E</td>
<td>(\lambda_E = ?)</td>
<td>(\lambda_E = 1.00) (\lambda_E = 1.00)</td>
</tr>
<tr>
<td>Acceleration, a</td>
<td>(\lambda_a = ?) (= 1/\lambda_l \cdot \lambda_E/\lambda_p)</td>
<td>(\lambda_a = 1.00) (\lambda_a = 1.00)</td>
</tr>
<tr>
<td>Density, (\rho)</td>
<td>(\lambda_p = \lambda_E/(\lambda_l \lambda_a)) (= ?)</td>
<td>(\lambda_p = 0.33) (\lambda_p = 1.00)</td>
</tr>
<tr>
<td>Velocity, (v)</td>
<td>(\lambda_v = \sqrt{\lambda_l \cdot \lambda_a})</td>
<td>(\lambda_v = 1.73) (\lambda_v = 1.73)</td>
</tr>
<tr>
<td>Forces, (f)</td>
<td>(\lambda_f = \lambda_E \lambda_l^2)</td>
<td>(\lambda_f = 9.00) (\lambda_f = 9.00)</td>
</tr>
<tr>
<td>Stress, (\sigma)</td>
<td>(\lambda_\sigma = \lambda_E)</td>
<td>(\lambda_\sigma = 1.00) (\lambda_\sigma = 1.00)</td>
</tr>
<tr>
<td>Strain, (\varepsilon)</td>
<td>(\lambda_\varepsilon = 1.00)</td>
<td>(\lambda_\varepsilon = 1.00) (\lambda_\varepsilon = 1.00)</td>
</tr>
<tr>
<td>Area, (A)</td>
<td>(\lambda_A = \lambda_l^2)</td>
<td>(\lambda_A = 9.00) (\lambda_A = 9.00)</td>
</tr>
<tr>
<td>Volume, (V)</td>
<td>(\lambda_A = \lambda_l^3)</td>
<td>(\lambda_A = 27.00) (\lambda_A = 27.00)</td>
</tr>
<tr>
<td>Second Moment of Area, (I)</td>
<td>(\lambda_I = \lambda_l^4)</td>
<td>(\lambda_I = 81.00) (\lambda_I = 81.00)</td>
</tr>
<tr>
<td>Mass, (m)</td>
<td>(\lambda_m = \lambda_p \lambda_l^3)</td>
<td>(\lambda_m = 9.00) (\lambda_m = 27.00)</td>
</tr>
<tr>
<td>Impulse, (i)</td>
<td>(\lambda_i = \lambda_l^2 \cdot \sqrt{\lambda_E/\lambda_p})</td>
<td>(\lambda_i = 15.59) (\lambda_i = 27.00)</td>
</tr>
<tr>
<td>Energy, (e)</td>
<td>(\lambda_e = \lambda_E \lambda_l^3)</td>
<td>(\lambda_e = 27.00) (\lambda_e = 27.00)</td>
</tr>
<tr>
<td>Frequency, (\omega)</td>
<td>(\lambda_\omega = 1/\lambda_l \cdot \sqrt{\lambda_E/\lambda_p})</td>
<td>(\lambda_\omega = 0.58) (\lambda_\omega = 0.33)</td>
</tr>
<tr>
<td>Time (Period), (t)</td>
<td>(\lambda_t = \sqrt{\lambda_l \cdot \lambda_a})</td>
<td>(\lambda_t = 1.73) (\lambda_t = 1.73)</td>
</tr>
<tr>
<td>Gravitational</td>
<td>(\lambda_g = 1.00)</td>
<td>(\lambda_g = 1.00) (\lambda_g = 1.00)</td>
</tr>
<tr>
<td>Acceleration, (g)</td>
<td>(\lambda_{fg} = \lambda_p \lambda_l^3)</td>
<td>(\lambda_{fg} = 9.00) (\lambda_{fg} = 27.00)</td>
</tr>
<tr>
<td>Gravitational Force, (f_g)</td>
<td>(\lambda_{fg} = \lambda_p \lambda_l^3)</td>
<td>(\lambda_{fg} = 9.00) (\lambda_{fg} = 27.00)</td>
</tr>
<tr>
<td>Critical Damping, (\xi)</td>
<td>(\lambda_\xi = 1.00)</td>
<td>(\lambda_\xi = 1.00) (\lambda_\xi = 1.00)</td>
</tr>
</tbody>
</table>

** Note for modeling with constant acceleration, \(\lambda_a\) becomes the independent variable \((= 1.00)\) and \(\lambda_p\) becomes the dependent variable \((= \lambda_E/\lambda_l)\).
APPENDIX B

INSTRUMENTATION

B 1. Load Cells

Special force transducers (load cells) to measure the internal force response of the model, which include axial loads, shear forces, and bending moments, were fabricated of mild steel and installed in the mid-story height of the first and second story columns and between fluid damper braces, shown in Fig.B-1 (designated by tag name LC# with measured force components N#, MX#, MY#, SX# and SY#). There were four actively wired load cells on the east side of the first and second story respectively, while there were four inactive ("dummy") load cells on the west side of the first and second story to maintain symmetry of stiffness in the model. The shear forces and bending moments were recorded in both the direction of motion and the transverse direction of motion. The load cells were designed such that the stiffness was similar to the concrete column.

Based on the yield strength of the steel, the axial, shear, and bending moment capacity ratings of the load cells are ±40 kips, ±5 kips, and ±40 kips-in respectively.

B 2. Displacement Transducers

Linear displacement sonic transducers (Temposonics™) were used to measure the absolute response displacements in the longitudinal (horizontal) direction of the base and each story level of the model during the shaking table tests. Fig.B-1 shows the location of the displacement transducers (designated by tag name D#) mounted on the east and west base and column-slab intersections on the north side of the model. The displacement transducers were also mounted between fluid damper braces to measure the displacement induced in dampers. The displacement transducers: have global displacement ranges of ±6 in., ±8 in., and ±10 in.; accuracy of ±0.05%
of the full scale displacement, 0.003, 0.004 and 0.005 in., respectively; were conditioned by a generic power supply and manufacturer amplifier-decoders; and were calibrated for the respective full scale displacement per 10 volts.

B 3. Accelerometers

Resistive accelerometers (Endevco™, ±25g) were used to measure the absolute story level accelerations of the model. Fig. 4-8) shows the location of each accelerometer with the respective tag name at the base, first, second, and third stories of the model in the direction of motion (designated by the name AH#), transverse to direction of motion (designated by tag name AT#), and for vertical motion (designated by tag name AV#). In the direction of motion, accelerometers were mounted on the east and west sides of the structure to detect any torsional response or out-of-phase motions. The accelerometers were conditioned with 2310 Vishay Signal Conditioning Amplifiers, which filtered frequencies above 25 Hz., calibrated for an acceleration range of ±2 g per 10 volts, and have nonlinearities of ±1.0% of the recorded acceleration.
Figure B-1  Instrumentation Identification and Locations
The National Center for Earthquake Engineering Research (NCEER) publishes technical reports on a variety of subjects related to earthquake engineering written by authors funded through NCEER. These reports are available from both NCEER’s Publications Department and the National Technical Information Service (NTIS). Requests for reports should be directed to the Publications Department, National Center for Earthquake Engineering Research, State University of New York at Buffalo, Red Jacket Quadrangle, Buffalo, New York 14261. Reports can also be requested through NTIS, 5285 Port Royal Road, Springfield, Virginia 22161. NTIS accession numbers are shown in parenthesis, if available.


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NCEER-87-0004 "The System Characteristics and Performance of a Shaking Table," by J.S. Hwang, K.C. Chang and G.C. Lee, 6/1/87, (PB88-134259). This report is available only through NTIS (see address given above).


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C-14


