Adaptive Control of Base Isolation Systems Using the Transmissibility-Based Semi-Active Controller

Ramin Rabiee

Old Dominion University, rrabi002@odu.edu

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ADAPTIVE CONTROL OF BASE ISOLATION SYSTEMS USING THE 
TRANSMISSIBILITY-BASED SEMI-ACTIVE CONTROLLER 

by 

Ramin Rabiee 

B.Sc. August 2010, Azad University 
M.Sc. July 2013, Shiraz University of Technology 

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OLD DOMINION UNIVERSITY 
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Approved by: 

Yunbyeong Chae (Director) 
Zia Razzaq (Member) 
Duc Nguyen (Member) 
Gene Hou (Member)
ABSTRACT

ADAPTIVE CONTROL OF BASE ISOLATION SYSTEMS USING THE TRANSMISSIBILITY-BASED SEMI-ACTIVE CONTROLLER

Ramin Rabiee
Old Dominion University, 2019
Director: Dr. Yunbyeong Chae

Base isolation system is one of the most effective seismic protection systems which is widely used to protect buildings of high importance against seismic hazards. However, the performance of these systems might be impaired under long-period earthquake ground motions due to the resonance effect. The main focus of this dissertation is the development of an adaptive semi-active control algorithm to improve the performance of conventional base isolation systems. The proposed control law works based on the transmissibility theory, called the transmissibility-based semi-active (TSA) controller, which can adaptively change the damping of the base-isolation system based on the response of the structure. Furthermore, a systematic design procedure was developed for the design of base-isolated structures with semi-active damping devices, which is called the simplified design procedure. The effectiveness of the proposed base isolation system with the TSA controller is investigated both numerically and experimentally, while using the simplified design procedure for design of the base-isolated buildings with magneto-rheological (MR) dampers. Nonlinear time history analyses are conducted with various long- and short-period earthquake ground motions to numerically evaluate the performance of the proposed base isolation system. Statistical analysis of the numerical simulation results are provided accordingly. Additionally, Real-Time Hybrid Simulations (RTHSs) are performed on a small-scale base-isolated building with an MR damper to experimentally validate the performance of the proposed base isolation system. It is shown that the proposed base isolation system makes the building work like a passive-on or passive-off isolation system as necessary to achieve high performance level under both long- and short-period earthquake ground motions, which can significantly improve the resiliency and sustainability of buildings.
This dissertation is dedicated to the love of my life, Sadena.
“To do life together” with you!
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# TABLE OF CONTENTS

LIST OF TABLES .................................................................................................................. viii

LIST OF FIGURES ............................................................................................................... ix

Chapter

1. INTRODUCTION ........................................................................................................... 1
   1.1 GENERAL BACKGROUND ....................................................................................... 1
   1.2 OBJECTIVES AND SCOPE ...................................................................................... 6
   1.3 ORGANIZATION OF DISSERTATION ......................................................................... 7

2. BASE ISOLATION SYSTEMS WITH MR DAMPERS AND SEMI-ACTIVE CONTROL LAWS ........................................................................................................ 10
   2.1 GENERAL ................................................................................................................ 10
   2.2 PASSIVE BASE ISOLATION SYSTEMS ...................................................................... 11
   2.3 MODELING OF BASE ISOLATORS .......................................................................... 12
   2.4 MR DAMPER MODELS ............................................................................................ 14
   2.5 SEMI-ACTIVE CONTROLLERS FOR BASE-ISOLATED STRUCTURES IMPLEMENTING MR DAMPERS .................................................................................................................. 20
   2.6 SUMMARY .............................................................................................................. 32

3. TRANSMISSIBILITY-BASED SEMI-ACTIVE (TSA) CONTROL ALGORITHM .......................................................................................................................... 34
   3.1 GENERAL .............................................................................................................. 34
   3.2 TRANSMISSIBILITY-BASED SEMI-ACTIVE (TSA) CONTROL LAW ...................... 35
   3.3 SUMMARY .............................................................................................................. 41

4. SIMPLIFIED DESIGN PROCEDURE FOR BASE-ISOLATED STRUCTURES WITH MR DAMPERS .............................................................................................................. 42
   4.1 GENERAL .............................................................................................................. 42
   4.2 SIMPLIFIED DESIGN PROCEDURE ....................................................................... 43
   4.3 SUMMARY .............................................................................................................. 55

5. NUMERICAL ASSESSMENT OF THE ADAPTIVE BASE ISOLATION ............ 56
   5.1 GENERAL .............................................................................................................. 56
   5.2 DESIGN OF A BASE-ISOLATED BUILDING WITH MR DAMPERS ........................ 56
   5.3 SELECTED EARTHQUAKE GROUND MOTIONS ..................................................... 60
# LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.1 Design Parameters for the Base Isolator</td>
<td>58</td>
</tr>
<tr>
<td>5.2 Design Parameters for the MR Damper</td>
<td>58</td>
</tr>
<tr>
<td>5.3 Design Results of Each Structural System</td>
<td>58</td>
</tr>
<tr>
<td>5.4 Long-period Earthquake Ground Motions</td>
<td>60</td>
</tr>
<tr>
<td>5.5 Short-period Earthquake Ground Motions</td>
<td>60</td>
</tr>
<tr>
<td>5.6 Parameters of the Base Isolator Based on the Bouc-Wen Hysteresis Model</td>
<td>62</td>
</tr>
<tr>
<td>5.7 Parameters for the MNS MR Damper Model</td>
<td>63</td>
</tr>
<tr>
<td>6.1 Coefficients of MR Damper and Rubber Bearing Models</td>
<td>88</td>
</tr>
<tr>
<td>6.2 Parameters for the MNS MR Damper Model</td>
<td>89</td>
</tr>
<tr>
<td>6.3 Parameters of the Bouc-Wen Model and the Columb’s Frictional Element</td>
<td>91</td>
</tr>
<tr>
<td>6.4 Long-period Earthquake Ground Motions</td>
<td>93</td>
</tr>
<tr>
<td>6.5 Short-period Earthquake Ground Motions</td>
<td>93</td>
</tr>
<tr>
<td>6.6 Natural Periods of the Small-scale Structure</td>
<td>96</td>
</tr>
<tr>
<td>6.7 Initial System Coefficients and Their Ranges for ATS Compensator</td>
<td>103</td>
</tr>
</tbody>
</table>
## LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1 Force-displacement Relationship of Different Isolators</td>
<td>12</td>
</tr>
<tr>
<td>2.2 Models of Base Isolators</td>
<td>13</td>
</tr>
<tr>
<td>2.3 Maxwell Nonlinear Slider (MNS) MR Damper Model</td>
<td>15</td>
</tr>
<tr>
<td>2.4 Force-velocity Relationship of Maxwell Element under Harmonic Motion</td>
<td>17</td>
</tr>
<tr>
<td>2.5 MR Damper Response; (a) Force-velocity, (b) Force-displacement Relationships</td>
<td>18</td>
</tr>
<tr>
<td>2.6 Schematic of Force-velocity Relationship of MR Damper under Sinusoidal Displacement</td>
<td>20</td>
</tr>
<tr>
<td>2.7 Block Diagram of a Passively-controlled Base-isolated Structure with MR Dampers</td>
<td>21</td>
</tr>
<tr>
<td>2.8 Block Diagram of a Semi-actively Controlled Base-isolated Structure with MR Dampers</td>
<td>22</td>
</tr>
<tr>
<td>3.1 Transmissibility of an SDOF System under Harmonic Excitation</td>
<td>37</td>
</tr>
<tr>
<td>3.2 Definition of Characteristic Period ($T_c$)</td>
<td>39</td>
</tr>
<tr>
<td>4.1 Bilinear Model Force-displacement Relationship</td>
<td>45</td>
</tr>
<tr>
<td>4.2 Simple Frictional Model for MR Dampers; (a) Force-displacement Relationship, (b) Force-velocity Relationship</td>
<td>47</td>
</tr>
<tr>
<td>4.3 Bingham Model for MR Dampers; (a) Force-displacement Relationship, (b) Force-velocity Relationship</td>
<td>48</td>
</tr>
<tr>
<td>4.4 Herschel-Bulkley Model for MR Dampers; (a) Force-displacement Relationship, (b) Force-velocity Relationship</td>
<td>49</td>
</tr>
<tr>
<td>4.5 Block Diagram of the Simplified Design Procedure for Base-isolated Structures with MR Dampers</td>
<td>54</td>
</tr>
<tr>
<td>5.1 Schematic of a Base-isolated Building with MR Dampers</td>
<td>57</td>
</tr>
<tr>
<td>5.2 Spectral Accelerations for Selected Long- and Short-period Earthquake Ground Motions</td>
<td>61</td>
</tr>
</tbody>
</table>
5.3 Block Diagram for Numerical Simulation of Adaptively Controlled Base-isolated Structure with MR Damper Controlled by the TSA Control Law .......................... 65

5.4 Responses of the Isolated Building under the 1979 Imperial Valley Earthquake (Long-period): (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift, (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration .................. 68

5.5 Base Floor Acceleration ($a_{base}$) Response of the Isolated Building with the Proposed Method under the 1979 Imperial Valley Earthquake ............................... 69

5.6 Comparison of Hysteresis Loops of the Base Isolator and the MR Damper for each Control Case under the 1979 Imperial Valley Earthquake ........................................ 69

5.7 Responses of the Isolated Building under the 1989 Loma Prieta Earthquake (Short-period): (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration .................. 72

5.8 Base Floor Acceleration ($a_{base}$) Response of the Isolated Building with the Proposed Method under the 1989 Loma Prieta Capitola Earthquake ............................... 73

5.9 Comparison of Hysteresis Loops of the Base Isolator and the MR Damper for each Control Case under the 1989 Loma Prieta Capitola Earthquake ............................... 73

5.10 Average of Maximum Structural Responses of Base-isolated Three-story Building. (a) under Long-period Ground Motions, (b) under Short-period Ground Motions .... 76

5.11 Comparison of Average Spectral Acceleration at the 3rd Floor of Building: (a) under Long-period Ground Motions, (b) under Short-period Ground Motions .......... 77

6.1 Schematic of the Structural System ................................................................. 82

6.2 Experimental Substructure (Base Isolation System) ........................................... 84

6.3 Characterization Tests: (a) Input Sine Wave, (b) Force-displacement Relationship of Rubber Bearing Combined with Linear Bearings, (c) Force-displacement Relationship of MR Damper under Maximum Input Current (i.e. 1.5 A) ...................... 86

6.4 Force-displacement Response of MR Damper under Sinusoidal Displacement Input for Various Constant Current Inputs (Frequency=0.1Hz, Amplitude=25mm) .......... 87

6.5 Base Isolation System Force-displacement Response under Sinusoidal Displacement Input (Frequency=0.1Hz, Amplitude=25mm): (a) 0 A to MR Damper (Passive-off System), (b) 1.5 A to MR Damper (Passive-on System) ........................................ 88

6.6 Linear-bearing Coulomb’s Friction Model ................................................................ 90
6.7 Comparison of Numerical Simulation Results with Experimental Test Results of the Combined Base Isolation System under the Sinusoidal Input Displacement (Input Current to the MR Damper = 1.5 A): (a) Input Sinusoidal Wave, (b) Force Time History, (c) Force-displacement Relationship ................................................................. 92

6.8 Response Spectra of the Original Earthquake Ground Motions ........................................ 93

6.9 Response Spectra of Scaled Earthquake Ground Motions .................................................... 95

6.10 Block Diagram for RTHS of the Semi-actively Controlled Base-isolated Structure with MR Damper .................................................................................................................... 99

6.11 Architecture of the Real-time Control System in the Structures Laboratory of Old Dominion University ............................................................................................................................. 102

6.12 Time History Responses under the 1979 Imperial Valley Earthquake (Short-period) for the Isolated Building: (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift, (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration ........................................................................................................................................... 105

6.13 Time History Responses under the 1976 Friuli, Italy Earthquake (Short-period) for the Isolated Building: (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift, (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration ........................................................................................................................................... 106

6.14 Base Floor acceleration \(a_{\text{base}}\) Response under the 1979 Imperial Valley Earthquake for the Isolated Building with the Proposed Controller ......................................................... 108

6.15 Base Floor Acceleration \(a_{\text{base}}\) Response under the 1976 Friuli, Italy Earthquake for the Isolated Building with the Proposed Controller ......................................................... 108

6.16 Hysteresis Loops of the Isolation System for Three Control Cases: (a) under the 1979 Imperial Valley Earthquake (Long-period), (b) under the 1976 Friuli, Italy Earthquake (Short-period) ..................................................................................................................................................................................... 109

6.17 Displacement Control Performance of the ATS Compensator during RTHS under the 1976 Friuli Earthquake: (a) Overall Displacement Tracking, (b) Synchronization Subspace Plot ......................................................... 110

6.18 Comparison of the RTHS and Numerical Simulation Results of the Proposed System under the 1994 Northridge Sylmar Long-period Earthquake: (a) Displacement Response, (b) Force Response, (c) Force-displacement Relationship ........................................ 112

6.19 Average of Maximum Structural Responses of Base-isolated Three-story Building: (a) under Long-period Ground Motions, (b) under Short-period Ground Motions .. 113
AA.1 Responses of the Isolated Building under the 1994 Northridge, Sylmar Earthquake (Long-period): (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration ..........133

AA.2 Base Floor Acceleration ($a_{base}$) Response of the Isolated Building with the Proposed Method under the 1994 Northridge Sylmar Converter (Long-period) ......................... 134

AA.3 Comparison of Hysteresis Loops of the Base Isolator and the MR Damper for each Control Case under the 1994 Northridge Sylmar Converter (Long-period) ............. 134

AA.4 Responses of the Isolated Building under the 1999 Chi-Chi, Taiwan Earthquake (Long-period): (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration ...... 135

AA.5 Base Floor Acceleration ($a_{base}$) Response of the Isolated Building with the Proposed Method under the 1999 Chi-Chi, Taiwan (Long-period) ........................................ 136

AA.6 Comparison of Hysteresis Loops of the Base Isolator and the MR Damper for each Control Case under the 1999 Chi-Chi, Taiwan (Long-period) ..................... 136

AA.7 Responses of the Isolated Building under the 1994 Northridge, Jensen Earthquake (Long-period): (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration ...... 137

AA.8 Base Floor Acceleration ($a_{base}$) Response of the Isolated Building with the Proposed Method under the 1994 Northridge, Jensen (Long-period) ......................... 138

AA.9 Comparison of Hysteresis Loops of the Base Isolator and the MR Damper for each Control Case under the 1994 Northridge, Jensen (Long-period) ....................... 138

AA.10 Responses of the Isolated Building under the 1990 Manjil, Iran Earthquake (Long-period): (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration ............... 139

AA.11 Base Floor Acceleration ($a_{base}$) Response of the Isolated Building with the Proposed Method under the 1990 Manjil, Iran (Long-period) ......................... 140

AA.12 Comparison of Hysteresis Loops of the Base Isolator and the MR Damper for each Control Case under the 1990 Manjil, Iran (Long-period) ........................... 140

AA.13 Responses of the Isolated Building under the 1994 Northridge, Canyon County Earthquake (Short-period): (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration ........................................................................141
<table>
<thead>
<tr>
<th>AA.14</th>
<th>Base Floor Acceleration ($a_{\text{base}}$) Response of the Isolated Building with the Proposed Method under the 1994 Northridge, Canyon County (Short-period)</th>
<th>142</th>
</tr>
</thead>
<tbody>
<tr>
<td>AA.15</td>
<td>Comparison of Hysteresis Loops of the Base Isolator and the MR Damper for each Control Case under the 1994 Northridge, Canyon County (Short-period)</td>
<td>142</td>
</tr>
<tr>
<td>AA.16</td>
<td>Responses of the Isolated Building under the 1992 Landers, Cool-water Earthquake (Short-period): (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration</td>
<td>143</td>
</tr>
<tr>
<td>AA.17</td>
<td>Base Floor Acceleration ($a_{\text{base}}$) Response of the Isolated Building with the Proposed Method under the 1992 Landers, Cool-water (Short-period)</td>
<td>144</td>
</tr>
<tr>
<td>AA.18</td>
<td>Comparison of Hysteresis Loops of the Base Isolator and the MR Damper for each Control Case under the 1992 Landers, Cool-water (Short-period)</td>
<td>144</td>
</tr>
<tr>
<td>AA.19</td>
<td>Responses of the Isolated Building under the 1994 Northridge, Beverly Hills Earthquake (Short-period): (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration</td>
<td>145</td>
</tr>
<tr>
<td>AA.20</td>
<td>Base Floor Acceleration ($a_{\text{base}}$) Response of the Isolated Building with the Proposed Method under the 1994 Northridge, Beverly Hills (Short-period)</td>
<td>146</td>
</tr>
<tr>
<td>AA.21</td>
<td>Comparison of Hysteresis Loops of the Base Isolator and the MR Damper for each Control Case under the 1994 Northridge, Beverly Hills (Short-period)</td>
<td>146</td>
</tr>
<tr>
<td>AA.22</td>
<td>Responses of the Isolated Building under the 1999 Kocaeli, Turkey Earthquake (Short-period): (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration</td>
<td>147</td>
</tr>
<tr>
<td>AA.23</td>
<td>Base Floor Acceleration ($a_{\text{base}}$) Response of the Isolated Building with the Proposed Method under the 1999 Kocaeli, Turkey (Short-period)</td>
<td>148</td>
</tr>
<tr>
<td>AA.24</td>
<td>Comparison of Hysteresis Loops of the Base Isolator and the MR Damper for each Control Case under the 1999 Kocaeli, Turkey (Short-period)</td>
<td>148</td>
</tr>
<tr>
<td>AB.1</td>
<td>Displacement Control Using the ATS Compensator under the 1979 Imperial Valley Earthquake (Long-period): (a) Overall Displacement Tracking, (b) Synchronization Subspace Plot</td>
<td>149</td>
</tr>
<tr>
<td>AB.2</td>
<td>Time History Responses under the 1994 Northridge Sylmar Convertor Earthquake (Long-period) for the Isolated Building: (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift, (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration</td>
<td>150</td>
</tr>
</tbody>
</table>
AB.3 Base Floor Acceleration ($a_{base}$) Response under the 1994 Northridge Sylmar Convertor Earthquake (Long-period) for the Isolated Building with the Proposed Controller .............................................................. 151

AB.4 Hysteresis Loops of the Isolation System for Three Control Cases under the 1994 Northridge Sylmar Convertor Earthquake (Long-period) .............................................................. 151

AB.5 Displacement Control using the ATS Compensator under the 1994 Northridge Sylmar Convertor (Long-period): (a) Overall Displacement Tracking, (b) Synchronization Subspace Plot ..................................................................................... 152

AB.6 Time History Responses under the 1999 Chi-Chi, Taiwan Earthquake (Long-period) for the Isolated Building: (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift, (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration .............................................................................................. 153

AB.7 Base Floor Acceleration ($a_{base}$) Response under the 1999 Chi-Chi, Taiwan Earthquake (Long-period) for the Isolated Building with the Proposed Controller... 154

AB.8 Hysteresis Loops of the Isolation System for Three Control Cases under the 1999 Chi-Chi, Taiwan Earthquake (Long-period) .............................................................. 154

AB.9 Displacement Control using the ATS Compensator under the 1999 Chi-Chi, Taiwan (Long-period): (a) Overall Displacement Tracking, (b) Synchronization Subspace Plot ..................................................................................... 155

AB.10 Time History Responses under the 1994 Northridge, Jensen Earthquake (Long-period) for the Isolated Building: (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift, (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration .............................................................................................. 156

AB.11 Base Floor Acceleration ($a_{base}$) Response under the 1994 Northridge, Jensen Earthquake (Long-period) for the Isolated Building with the Proposed Controller. 157

AB.12 Hysteresis Loops of the Isolation System for Three Control Cases under the 1994 Northridge, Jensen Earthquake (Long-period) .............................................................. 157

AB.13 Displacement Control using the ATS Compensator under the 1994 Northridge, Jensen (Long-period): (a) Overall Displacement Tracking, (b) Synchronization Subspace Plot ..................................................................................... 158

AB.14 Time History Responses under the 1990 Manjil, Iran Earthquake (Long-period) for the Isolated Building: (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift, (d) 3rd Floor Absolute Velocity, And (e) 3rd Floor Absolute Acceleration .............................................................................................. 159
AB.15 Base Floor Acceleration ($a_{\text{base}}$) Response under the 1990 Manjil, Iran Earthquake (Long-period) for the Isolated Building with the Proposed Controller .......... 160

AB.16 Hysteresis Loops of the Isolation System for Three Control Cases under the 1990 Manjil, Iran Earthquake (Long-period) .................................................................................................................. 160

AB.17 Displacement Control using the ATS Compensator under the 1990 Manjil, Iran (Long-period): (a) Overall Displacement Tracking, (b) Synchronization Subspace Plot ............................................................................................................. 161

AB.18 Time History Responses under the 1989 Loma Prieta, Capitola Earthquake (Short-period) for the Isolated Building: (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift, (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration .................................................................................................................. 162

AB.19 Base Floor Acceleration ($a_{\text{base}}$) Response under the 1989 Loma Prieta, Capitola Earthquake (Short-period) for the Isolated Building with the Proposed Controller... 163

AB.20 Hysteresis Loops of the Isolation System for Three Control Cases under the 1989 Loma Prieta, Capitola Earthquake (Short-period) .................................................. 163

AB.21 Displacement Control using the ATS Compensator under the 1989 Loma Prieta, Capitola Earthquake (Short-period): (a) Overall Displacement Tracking, (b) Synchronization Subspace Plot ............................................................................................................. 164

AB.22 Time History Responses under the 1971 San Fernando, LA Hollywood Earthquake (Short-period) for the Isolated Building: (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift, (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration .................................................................................................................. 165

AB.23 Base Floor Acceleration ($a_{\text{base}}$) Response under the 1971 San Fernando, LA Hollywood Earthquake (Short-period) for the Isolated Building with the Proposed Controller .......................................................... 166

AB.24 Hysteresis Loops of the Isolation System for Three Control Cases under the 1971 San Fernando, LA Hollywood Earthquake (Short-period) .................................................. 166

AB.25 Displacement Control using the ATS Compensator under the 1971 San Fernando, LA Hollywood Earthquake (Short-period): (a) Overall Displacement Tracking, (b) Synchronization Subspace Plot ............................................................................................................. 167

AB.26 Time History Responses under the 1992 Cape Mendocino, Rio Del Overpass Earthquake (Short-period) for the Isolated Building: (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift, (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration .................................................................................................................. 168
AB.27 Base Floor Acceleration ($a_{base}$) Response under the 1992 Cape Mendocino, Rio Del Overpass Earthquake (Short-period) for the Isolated Building with the Proposed Controller ................................................................. 169

AB.28 Hysteresis Loops of the Isolation System for Three Control Cases under the 1992 Cape Mendocino, Rio Del Overpass Earthquake (Short-period) ........................... 169

AB.29 Displacement Control using the ATS Compensator under the 1992 Cape Mendocino, Rio Del Overpass Earthquake (Short-period): (a) Overall Displacement Tracking, (b) Synchronization Subspace Plot................................................................. 170

AB.30 Time History Responses under the 1994 Northridge, Canyon County Earthquake (Short-period) for the Isolated Building: (a) Ground Acceleration, (b) Base Isolator Displacement, (c) 3rd Story Drift, (d) 3rd Floor Absolute Velocity, and (e) 3rd Floor Absolute Acceleration................................................................. 171

AB.31 Base Floor Acceleration ($a_{base}$) Response under the 1994 Northridge, Canyon County Earthquake (Short-period) for the Isolated Building with the Proposed Controller ................................................................. 172

AB.32 Hysteresis Loops of the Isolation System for Three Control Cases under the 1994 Northridge, Canyon County Earthquake (Short-period) ........................... 172

AB.33 Displacement Control using the ATS Compensator under the 1994 Northridge, Canyon County Earthquake (Short-period): (a) Overall Displacement Tracking, (b) Synchronization Subspace Plot................................................................. 173

AB.34 Comparison of the RTHS and Numerical Simulation Results of Proposed System under the 1990 Manjil, Iran (Long-period): (a) Displacement Response, (b) Force Response, (c) Force-Displacement Relationship .................................................. 174

AB.35 Comparison of the RTHS and Numerical Simulation Results of Proposed System under the 1994 Northridge, Jensen (Long-period): (a) Displacement Response, (b) Force Response, (c) Force-Displacement Relationship .................................................. 174

AB.36 Comparison of the RTHS and Numerical Simulation Results of Proposed System under the 1999 Chi-Chi, Taiwan Earthquake (Long-period): (a) Displacement Response, (b) Force Response, (c) Force-displacement Relationship ......................... 175

AB.37 Comparison of the RTHS and Numerical Simulation Results of Proposed System under the 1979 Imperial Valley earthquake (Long-period): (a) Displacement Response, (b) Force Response, (c) Force-displacement Relationship ......................... 175

AB.38 Comparison of the RTHS and Numerical Simulation Results of Proposed System under the 1989 Loma Prieta, Captolala Earthquake (Short-period): (a) Displacement Response, (b) Force Response, (c) Force-displacement Relationship ......................... 176
AB.39 Comparison of the RTHS and Numerical Simulation Results of Proposed System under the 1976 Friuli, Italy Earthquake (Short-period): (a) Displacement Response, (b) Force Response, (c) Force-displacement Relationship ........................................... 176

AB.40 Comparison of the RTHS and Numerical Simulation Results of Proposed System under the 1992 Cape Mendocino, Rio Del Overpass Earthquake (Short-period): (a) Displacement Response, (b) Force Response, (c) Force-displacement Relationship .............................................................................................................. 177

AB.41 Comparison of the RTHS and Numerical Simulation Results of Proposed System under the 1994 Northridge, Canyon County Earthquake (Short-period): (a) Displacement Response, (b) Force Response, (c) Force-displacement Relationship .............................................................................................................. 177

AB.42 Comparison of the RTHS and Numerical Simulation Results of Proposed System under the 1971 San Fernando, LA Hollywood Earthquake (Short-period): (a) Displacement Response, (b) Force Response, (c) Force-displacement Relationship .............................................................................................................. 178
CHAPTER 1

INTRODUCTION

1.1 General Background

During the lifetime of civil infrastructures such as bridges, highways and buildings, they might be subjected to extreme loading. Earthquake loading is one kind of extreme loads which has already shown its catastrophic effect through damaging earthquakes such as 1994 Northridge, 1979 Imperial Valley, 1990 Manjil, etc. It is observed that not only an earthquake can bring huge losses to resources, but also it might cause many fatalities to human lives. These incidents have proven the importance of seismic design for structures to structural engineers. During the previous couple of decades, many changes have been made to structural design codes to provide the best performance of seismic protection systems. Among the protection systems, base isolation, which can effectively reduce the earthquake-induced forces to structural and non-structural components, is one of the most efficient systems.

Base isolation systems provide a low lateral stiffness layer between the base and foundation of the structure. This will increase the natural period of the base-isolated structure in comparison to the same fixed-base structure with no base isolators. At longer periods, it can be clearly observed that the spectral acceleration of a structure will be decreased as the natural period increases (e.g. ASCE7-10 [1]). Thus, the increase in the natural period of base-isolated structures will be very effective not only in the reduction of the acceleration of non-structural components but also in the reduction of design forces of the structural elements above the isolation layer, making the base isolation system effective for seismic hazard mitigation. Due to the low lateral stiffness of the isolation layer, the
deformation at the base of the structure will be large. This issue will be of higher importance especially once an earthquake with long-period contents occurs which increases the possibility of resonance, resulting in larger displacements of base isolators [2]. The deformation provided by the long-period earthquake ground motions due to the large ground velocities and displacements can exceed the displacement capacity of the isolator or even the gap width between the moat wall and the building. This can introduce a large impact force to the structure and pounding effects which can threaten the stability of the whole structural system and finally cause collapse or serious damage [3-5]. Full-scale shake table tests also have shown the vulnerability of conventional base-isolation systems under long-period ground motions [6, 7], where non-structural components such as medical and service equipment and furniture were severely damaged by long-period earthquakes. Thus, the performance of conventional base isolation systems needs to be improved under long-period earthquakes.

Supplemental damping devices can be effective tools for enhancing the performance of base-isolated structures under long-period earthquake ground motions. However, it is observed that increasing damping under short-period earthquake ground motions will increase the structural responses above the isolation layer especially in terms of story drifts and accelerations [8, 9]. It is therefore beneficial to use energy dissipation systems that can adaptively change the damping. Damping systems can be categorized in three different groups, i.e., passive, active and semi-active damping systems.

Passive dampers generally work based on principles such as yielding of metals, deformation of visco-elastic (VE) solids or fluids, phase transformation in metals, frictional sliding, fluid orificing, etc. [10]. Tuned Mass and triangular-plate added damping and
stiffness (TADAS) dampers are examples of passive damping systems used for vibration control of structures, bridges and tall buildings [11-19]. Three approaches of data-driven, model-driven and model-data fusion have been developed by researchers for vibration-based structural monitoring of instrumented buildings [20-22]. The motion of passive dampers is passively determined by the motion of structures. The biggest advantage of passive damping systems is the stability of the structural system where they are implemented, making these systems easily acceptable among the design community [23]. However, passive systems cannot adaptively adjust the damping of the system.

Active damping systems incorporate actuators to deliver the required damping force with the aid of real-time processing controllers and sensors [24]. These systems are able to adjust the damping of the system through the user-defined controllers and structural control objectives. However, these systems incorporate different actuators that need huge power sources, sensors and control models, which makes their implementation very expensive. Actually, depending on the implemented control algorithm, the number of required sensors differs; however, practical considerations usually preclude the availability of a complete set of sensor information [25]. Accordingly, the optimization of sensors and controller locations is another challenging issue for active control systems. In addition, structural systems under external loads usually undergo nonlinear and time-dependent degrading behavior, which results in a performance that might be different from what was expected at the design stage of controllers; this is directly related to parameter uncertainties and system identification errors. Thus, the limited number of applied sensors and uncertainties related to the structural properties can provide control signals to actuators,
which might not be accurate enough, increasing the possibility of instability in the structural system depending on the control error.

Semi-active damping systems take advantage of both active and passive systems. The structural system with semi-actively controlled damping devices is able to adjust the damping of the system with a power consumption much lower than the active systems. Semi-active systems do not have the stability issue since they do not add any energy to the structural system. Smart tuned mass dampers, controllable fluid dampers, and variable-orifice fluid dampers are examples of semi-active damping devices that can be used in bridge or building structural systems [23, 26].

The magneto-rheological (MR) damper is one of the most popular semi-active damping devices [27-30]. MR dampers generally have cylindrical shapes with a piston which moves through a cylinder. The piston head divides the cylinder into two separate chambers that are filled with MR fluid. An electromagnetic coil is embedded in the piston head around the annular gap through which the MR fluid moves between chambers. The MR fluid contains iron particles that are aligned when the magnetic flux is provided, increasing the resistance of fluid movement and damping force. Once the magnetic field is removed, the MR fluid goes back to its original state. MR dampers need small power supplies and can easily switch between the two states, making them ideal semi-active devices for structural control.

Numerous studies have been conducted on the application of semi-active control devices during the last couple of decades, specifically for the application of these systems with base-isolated systems [31-39]. In spite of these existing studies, however, the practical application of semi-actively controlled base isolation systems is quite limited, which is
primarily related to the lack of knowledge for quantitatively predicting the response of the semi-actively controlled base isolation system under earthquake loads [8]. Existing control laws for the semi-active devices are often based on control objectives that are not directly related to minimizing the maximum structural responses of interest (e.g., the isolator displacement, story drifts, floor accelerations) [40]. In addition, the application of semi-active control laws into practical design methods is of most importance which has not been addressed in previously conducted research. The designers also should have an insight into the number and type of the semi-active devices required to be used in order to satisfy a specified performance objective for the structure. Not only do the previous studies not provide a systematic design procedure for base-isolated structures with semi-active damping devices but also the performance of existing methods under long-period earthquakes have not been validated accurately.

In this dissertation, the dynamic behavior of base-isolated structures with MR dampers is investigated. A new control algorithm is developed to effectively control the damping of the base-isolated structures for the purpose of seismic hazard mitigation under both long- and short-period earthquake ground motions. The new control law is based on the transmissibility theory of an isolated structure and can adaptively change the damping characteristics of the system based on the period contents in the response of structures. The new control law is called the Transmissibility-based Semi-Active (TSA) controller in this dissertation. Additionally, by implementing the TSA controller, a practical design procedure becomes feasible which can predict the response of the base-isolated structures with semi-active damping devices without conducting a complicated nonlinear time history analysis; this method is called the simplified design procedure in this dissertation. This
procedure includes simple models for MR dampers and base isolators. The procedure enables the prediction of the maximum base shear and displacement at the base level, which is necessary for the design of the superstructure. The simplified design procedure developed in this dissertation is implemented for the design of full-scale and small-scale base-isolated structures to numerically and experimentally evaluate the performance of the newly-developed semi-active control law. Intensive numerical simulations are conducted for the designed full-scale structure subjected to various earthquake ground motions, where sophisticated nonlinear numerical models for MR dampers and base isolators are used. Real-time hybrid simulations (RTHS) are conducted for the small-scale structure to experimentally assess the performance of the TSA controller. In the RTHS study, the base isolation system consisting of a rubber bearing, an MR damper and linear bearings is experimentally tested (i.e., the experimental substructure), while the structure above the isolation layer is modeled numerically (i.e., the analytical substructure). The following section describes the objectives of this dissertation.

1.2 Objectives and Scope

The objectives of this dissertation are as follows:

- Development of a new adaptive semi-active control system for base-isolated structures which can provide structural resiliency under both long- and short-period earthquake ground motions (i.e., the TSA controller)
- Development of a simplified design procedure for base-isolated structures with supplemental semi-active damping devices
- Numerical evaluation of the performance of the TSA controller for base-isolated structures with MR dampers through nonlinear time history analysis with different long- and short-period earthquake ground motions
- Conduction of real-time hybrid simulations to experimentally assess the performance of base-isolated structures with MR dampers controlled by the TSA controller.

1.3 Organization of Dissertation

- **Chapter 2** provides the background information of the base isolation system and its effective performance as a seismic protection system. Description of the general performance of MR dampers as one of the most well-known semi-active damping devices is also provided. Detailed information of the Bouc-Wen and Maxwell nonlinear slider (MNS) models for simulating the performance of base isolators and MR dampers, respectively, is presented. The fundamental theories of semi-active control systems along with various semi-active control algorithms for base-isolated structures with MR dampers are provided.

- **Chapter 3** develops a new semi-active control law for semi-actively controlled base-isolated structures. The new control law works based on the transmissibility theory and is named the *Transmissibility-based Semi-Active* (TSA) controller. The TSA controller enables a systematic design procedure for base-isolated structures with semi-active damping devices. Detailed descriptions about the TSA controller are provided in this chapter.

- **Chapter 4** develops a systematic design procedure for semi-actively controlled base-isolated structures based on the capabilities provided by the TSA control law.
The proposed procedure is called *Simplified Design Procedure* which is able to predict the behavior of base-isolated structures with MR dampers. Details about this procedure and implemented models for base isolators and MR dampers are also provided. This procedure is used for the design of full-scale and small-scale three-story base-isolated structures with MR dampers to evaluate the performance of the TSA controller numerically and experimentally in Chapters 5 and 6, respectively.

**Chapter 5** evaluates the performance of the TSA controller through numerical simulations for a full-scale three-story base-isolated structure with MR dampers designed using the simplified design procedure. Sophisticated numerical models which can consider the nonlinearities of the MR damper and base isolator (i.e., MNS and Bouc-Wen models, respectively) are used. Nonlinear time history analyses, as well as their statistical results, are conducted under different long- and short-period earthquake ground motions.

**Chapter 6** experimentally validates the performance of the TSA controller through real-time hybrid simulation tests. The architecture of the real-time control system at Old Dominion University is described. The analytical and experimental substructures of the RTHS tests are explained in detail. Characterization tests for a small-scale rubber bearing and a small-scale MR damper with various current inputs are conducted. A small-scale three-story building is designed using the simplified design procedure proposed in Chapter 3 which is subjected to various long- and short-period earthquake ground motions. The RTHS results are also compared with numerical simulation results, where the experimental substructure is replaced with its numerical models.
Chapter 7 provides the conclusions of findings in this study and makes recommendations for future study.
CHAPTER 2

BASE ISOLATION SYSTEMS WITH MR DAMPERS AND SEMI-ACTIVE
CONTROL LAWS

2.1 General

The effective performance of the conventional base-isolation systems in reducing
the superstructure responses is well observed during previous earthquakes but with
increased displacement especially under near-fault earthquake ground motions [41-45].
Many studies have been conducted during the last couple of decades to effectively reduce
the base isolator displacement. In most of these studies, researchers were trying to increase
the damping of the base-isolated structure since large damping is effective in reducing the
displacement. Damping of a system can be increased passively by adding specific elements
or materials to a base isolator such as lead plugs, or using control devices such as active or
semi-active devices [23, 42, 43, 46-59].

Active control of base isolation systems requires implementation of actuators to
apply the control force to a structure. Semi-active control of base isolation systems can be
achieved by implementing different semi-active damping devices such as smart tuned mass
dampers, controllable fluid dampers, variable-orifice fluid dampers, etc. Magneto-
rheological (MR) dampers are one of the most well-known semi-active damping devices
where the feasibility of using them as devices for the vibration reduction of base-isolated
structures has been studied by many researchers [33, 37, 38, 50, 60-78].

In this chapter, previous research related to base isolation systems, different models
for MR dampers, and semi-active control algorithms implemented for base isolation
systems is provided.
2.2 Passive Base Isolation Systems

Base isolation systems provide a low lateral stiffness layer between the base and foundation of the structure. It is intended to increase the natural period of the structure so that the acceleration response and seismic lateral force induced to the structure are effectively decreased. Several isolation methods can be implemented to achieve this goal. One of the most common isolators is the elastomeric bearing with a cylindrical shape which consists of hard rubber and steel shims. The steel shims are used to provide vertical strength and the rubber provides restoring force under lateral excitations. Since damping is very effective in reducing the deformation, lead plugs can be inserted into the bearings to increase the damping characteristics. Lead rubber bearings (LRB) can provide large hysteretic performance for dissipating energy from earthquake ground motions. Another popular type of isolation system is based on sliding elements between the foundation and the base of the structure, where the sliding displacements are controlled by high-tension springs or laminated rubber bearings, or by making the sliding surface curved [79]. The friction pendulum bearing (FPB) is one of these types of bearings where the weight of the structures is supported on spherical sliding surfaces providing restoring force by slightly raising the structure once sliding occurs. The force-displacement relationships of these isolators are shown in Figure 2.1.
2.3 Modeling of Base Isolators

Various structural elements can be used to numerically model base isolators. Elastic, viscous, and hysteretic elements can be used for modeling bilinear elastomeric elements. For modeling of slide bearings, hysteretic elements can be used. If the behavior of the elastomeric bearing is linear, linear elastic elements can be used for its modeling. The hysteretic behavior of base isolators can be divided into two parts, i.e., the pre-yield and post-yield modes. The base isolator model should be able to account for the initial stiffness of the pre-yield mode and the yielding stiffness of the post-yield mode. The bilinear and Bouc-Wen [81] models are the most well-known models typically used to model the hysteretic behavior of base isolators as shown in Figure 2.2 [33].
The restoring force from the base isolator is represented by $f_{BI}$ as:

$$f_{BI} = Q_h + k_y u_b$$  \hspace{1cm} (2.1)

$Q_h$ is the hysteretic force given as:

$$Q_h = z Q_{pb}$$  \hspace{1cm} (2.2)

where,

$$Q_{pb} = \left(1 - \frac{k_y}{k_i}\right) Q_y$$  \hspace{1cm} (2.3)

The initial stiffness is represented by $k_i$ and defined as the ratio of force to displacement at the yielding point (i.e. $Q_y/u_y$). The evolutionary variable of $z$ can be obtained by solving the following differential equation:

$$\dot{z} = - \gamma z |\dot{u}_b| |z|^{m-1} - \beta \dot{u}_b |z|^m + A u_b$$  \hspace{1cm} (2.4)

where $u_b$ and $\dot{u}_b$ are the displacement and velocity of the isolator, respectively. Also, $\gamma$, $\beta$, $A$, and $m$ are time-invariant shape parameters of the Bouc-Wen hysteresis model. It should
be noted that the Bouc-Wen model provides results more consistent with the experimental data [82] in comparison to the bilinear model, specifically due to the smooth transition from \( k_i \) to \( k_y \) or from \( k_y \) to \( k_i \). Actually, the evolutionary variable of \( z \) is included in the Bouc-Wen model to generate such a smooth hysteretic transition, providing better estimation of accelerations for base isolated structures [33].

2.4 MR Damper Models

The Bingham model is one of the simple models for describing the behavior of MR dampers. The damping force generated by this model is dependent on the damper velocity. This model consists of dashpot and friction elements connected in parallel, as first presented by Stanway et al. for electro-rheological (ER) vibration dampers [83, 84]. Gamota and Filisko’s model [85] was originally proposed for electro-rheological (ER) dampers, but Spencer et al. [86] implemented this model for modeling of an MR damper. This model is a combination of the Bingham model and a standard model for a linear solid connected in series. A combined model based on the Maxwell element and the Bingham model was proposed by Makris et al. [87] and named the BingMax model. This model was shown to be able to represent both the frequency-dependent and hysteretic behavior of an MR damper. A more complicated model for MR dampers was developed by Spencer et al. [86]. This model is a combination of dashpots and springs with the Bouc-Wen element and is able to capture the force roll-off phenomenon that occurs near zero velocity in an MR damper. This model is widely used in the numerical simulation of structures with the MR damper. Also, this model was used for modeling the first generation of large-scale MR dampers manufactured by the Lord Corporation [88]. This model is not able to account for the non-Newtonian behavior of the MR fluid, and the initial values for model parameters
should be identified by trial and error. The hyperbolic tangent model developed by Gavin [89] was used for modeling the electro-rheological (ER) dampers. This model consists of a series of Voigt visco-elastic elements combined with a lumped mass element and a nonlinear friction element. Bass and Christenson [90] used the hyperbolic tangent model for modeling the second generation of large-scale MR dampers manufactured by the Lord Corporation. Chae [29] developed a new MR damper model named the Maxwell Nonlinear Slider (MNS) model which was used to model large-scale MR dampers subjected to realistic earthquake displacement and velocity demands. The MNS MR damper model can independently describe the pre-yield and post-yield behavior of an MR damper, which makes it easier to identify the parameters for the model. The Herschel-Bulkley visco-plastic element [29, 91, 92] is incorporated into the MNS model for the description of the post-yield mode of the damper so that the non-Newtonian MR fluid property can be effectively accounted for. Due to the proven accuracy provided by the MNS model [93-95], it is considered for numerical modeling of MR dampers in this dissertation. Figure 2.3 shows the mechanical model of the MNS MR damper model.

Figure 2.3 Maxwell Nonlinear Slider (MNS) MR damper model [29]
In Figure 2.3, \( x \) is the degree of freedom of the model that is associated with the displacement of the MR damper piston relative to its initial position, while \( y \) and \( z \) are variables associated with the pre-yield mode of the model. \( x \) and \( \dot{x} \) are referred to as the damper displacement and the damper velocity, respectively. The damper force in the pre-yield mode is described by the Maxwell element consisting of a dashpot with coefficient \( c \) and stiffness \( k \) (see Figure 2.3) where \( f \) can be determined by solving the following differential equation

\[
f = k(y - z) = c \dot{z}
\]  

(2.5)

When the damper is in pre-yield mode, \( \dot{y} \) is equal to the damper velocity \( \dot{x} \). The initial value of \( y \) is set to be equal to \( x \); thus, equation (2.5) can be solved in terms of \( z \) for a given \( x \), enabling the damper force \( f \) to be determined. The parameters for the Maxwell element can be easily estimated from the visco-elastic behavior of the MR damper, especially when the damper is subjected to small displacement amplitudes and low velocities. The values of \( c \) and \( k \) of the Maxwell element are obtained from the force-velocity relationship by selecting two appropriate points on the hysteretic force-velocity curve and then applying visco-elasticity theory. Assuming the Maxwell element is subjected to a harmonic motion with an amplitude of \( x_0 \) and a circular excitation frequency of \( \omega \), the coefficients \( c \) and \( k \) are calculated as

\[
c = \frac{1}{x_0 \omega} \frac{f_0^2 + f_m^2}{f_m}, \quad k = \frac{1}{x_0 \omega} \frac{f_0^2 + f_m^2}{f_0}
\]  

(2.6)

where \( f_0 \) and \( f_m \) are the damper forces when the velocities are zero and maximum, respectively. Figure 2.4 illustrates the force-velocity relationship of the Maxwell element under a harmonic excitation and the definition of its coefficients.
The post-yield behavior of the MR damper is directly related to the velocity $\dot{x}$. The non-Newtonian behavior that occurs in the MR fluid can be well-described by the Herschel-Bulkley visco-plasticity theory. This means that the shear thinning and shear thickening behavior of the fluid can be described using a power law model [91, 92, 96]. In the MNS model, the post-yield response of the MR damper is modeled using the nonlinear slider of Figure 2.3. The post-yield curve in the force-velocity response of the MR damper consists of a linear line and a curve based on the Herschel-Bulkley model. The curved lines are tangential to the linear lines at velocities of $\dot{x}_t^+$ and $\dot{x}_t^-$ for the positive and negative velocities, respectively. The force-velocity and force-displacement relationships for both pre-yield and post-yield modes are shown in Figure 2.5. The negative force for the post-yield curve can be represented as
\[
    f_{py}(\dot{x}) = \begin{cases} 
    a + b|\dot{x}|^n & \text{if } \dot{x} \geq \dot{x}_t^- \\
    a_t(\dot{x} - \dot{x}_t^-) + f_t^- & \text{if } \dot{x} < \dot{x}_t^-
    \end{cases}
\]

where \(\dot{x}_t^-\), \(a\), \(b\), and \(n\) are the parameters of the model which should be identified by conducting experimental characterization tests, and \(a_t = bn|\dot{x}_t^-|^{n-1}\), \(f_t^- = a + b|\dot{x}_t^-|^n\). It should be noted that identification of the post-yield parameters are completely independent from identification of pre-yield coefficients \(c\) and \(k\) from equation (2.6). The positive force of the post-yield curve \(f_t^+\) can be found in the same way as \(f_t^-\).

Due to the imperfections in the manufacturing of MR dampers, the force-velocity response might not be symmetric. This issue has been observed in the large-scale MR dampers [93, 94]. Thus, an increasing phase and decreasing phase of the post-yield curve is defined depending on the changes in the force value (i.e. the damper is in an increasing phase if the force increases, and in a decreasing phase if the force decreases), as shown in Figure 2.6. Since the acceleration shows the rate of changes in the velocity, the damper is in the increasing phase once the acceleration (i.e. \(\ddot{x}\)) is greater than zero in the positive
force post-yield mode, while it is in the decreasing phase once \( \ddot{x} \) is lower than zero. In order to consider the discrepancy between the increasing and decreasing phases, a virtual mass of \( m_0 \) is considered which is added to the force value in the decreasing phase for both positive and negative forces as

\[
f (\text{Positive Force}) = \begin{cases} 
   f_{py}^+(\dot{x}) & \ddot{x} \geq 0; \text{increasing phase} \\
   f_{py}^+(\dot{x}) + m_0 \dot{x} & \ddot{x} < 0; \text{decreasing phase}
\end{cases}
\]

\[
f (\text{Negative Force}) = \begin{cases} 
   f_{py}^-(\dot{x}) & \ddot{x} \geq 0; \text{increasing phase} \\
   f_{py}^-(\dot{x}) + m_0 \dot{x} & \ddot{x} < 0; \text{decreasing phase}
\end{cases}
\]

(2.8)

The transition from pre-yield to post-yield occurs once the force from the Maxwell element reaches the post-yield curve, and the nonlinear slider becomes activated thereafter. This condition can be mathematically represented as

\[
|f| = |f_{py}(\dot{x})|
\]

(2.9)

where \( f_{py} \) is associated with either positive or negative force post-yield curves. The transition from the post-yield mode to the pre-yield mode occurs once \( \dot{x} \) equals to \( \dot{y} \), where

\[
\dot{y} = \frac{f}{k} + \frac{f}{c}
\]

In order to have a smooth transition from the post-yield mode to the pre-yield mode, the pre-yield mode variables \( y \) and \( z \), are continuously updated during the post-yield mode.
2.5 Semi-Active Controllers for Base-Isolated Structures Implementing MR Dampers

The implementation of MR dampers as semi-active control devices for base-isolation systems has been of interest to many researchers due to their applicability to base isolators. MR dampers can be used either in semi-active or passive mode for controlling the response of base-isolated structures. In the passive control mode, a constant current is applied to the damper and no feedback data is required from sensors attached to the structure. Generally, if the current applied to the damper is maximum, the system is called \textit{passive-on} system, and the system with minimum current is called \textit{passive-off} system. The block diagram of the base-isolated structure with passive MR dampers is shown in Figure 2.7.
In the semi-active controlled base-isolation systems with MR dampers, sensors and a controller are required so that the feedback data from the sensors is appropriately implemented within the controller to effectively reduce the vibration of the base-isolation systems. The controller makes a decision based on the feedback data, providing an optimal command current be applied to the MR damper. Depending on the applied current to the MR damper, the amount of damping force will be specified. It should be noted that, in contrast to active systems, semi-active systems do not add any energy to the system, where the damper force is used for dissipating the energy. Actually, changing the current to the damper does not change the direction of the damping force, like an active system, and just the amount of force will be changed. Determination of the command current will be based on the feedback data collected from different sensors attached to the structural system, such as accelerometers, load cells, displacement transducers, etc. as well as the given semi-active controller. Figure 2.8 shows the block diagram of a semi-actively controlled base-isolated structure with MR dampers.
The command current sent by the controller to the damper can be continuous or a simple on-off type command with either maximum or minimum current. Due to this capability of MR dampers to be controlled by adjusting the input current, many researchers have studied the performance of the combined base-isolated structures with MR dampers. Additionally, benchmark base-isolated building models have been also developed [80, 97] so that the performance of different semi-active controllers for MR dampers can be evaluated [60]. In this section, various semi-active controllers applied to base-isolated structures with MR dampers are briefly introduced.

2.5.1 Base Isolation Systems Implementing Lyapunov Stability Control Theory

Lyapunov stability theory provides a means of stabilizing unstable nonlinear systems using feedback control [98]. The whole idea is that if a suitable Lyapunov function is selected and forced to decrease along the trajectories of the system, the resulting system will converge to its equilibrium. Providing a powerful tool for solving the stability problems in both linear and nonlinear systems, the Lyapunov stability theory has been used
as the base for designing many controllers such as linear quadratic regulator and the sliding mode controllers [99].

Sahasrabudhe and Nagarajaih [50] conducted experimental and analytical studies on a base-isolated bridge model having sliding bearings combined with an MR damper. For the experimental studies, they performed shake table tests on a 1:20 scaled bridge model, and the MR damper was installed between the bridge deck and pier, where the experimental setup was subjected to several near-fault earthquake ground motions. A Lyapunov control algorithm was developed for the semi-active control of the MR damper based on the total strain energy, the total dissipated energy and the total kinetic energy. Results of the analytical and shake table tests were also compared. It was observed that their semi-active control system with MR damper reduced the bearing displacements further than the cases of passive low- and high-damping, while maintaining isolation level forces less than the passive high-damping case.

2.5.2 Base Isolation Systems Implementing Linear Quadratic Regulator (LQR) and Linear Quadratic Gaussian (LQG) Controllers

Linear quadratic regulator (LQR) is an optimal control regulator which uses a feedback controller to minimize a global criterion/quadratic cost function. Linear Quadratic Gaussian (LQG) determines an output feedback law that is optimal in minimizing the expected value of a quadratic cost criterion. LQR/LQG methods have been frequently used to control the response of base-isolated structures with MR dampers. Spencer et al. [75] used the clipped-optimal controller based on the H2/LQG developed by Dyke et al. [27] for a linear, two-degree-of-freedom, lumped-mass model of a base-isolated building equipped with MR dampers. The smart isolation system they suggested was shown to be a
most effective alternative for a broad class of earthquakes including near-source events. Yoshioka et al. [37] conducted experimental studies on an adaptive base-isolated two-degree-of-freedom building model equipped with a sponge-type MR damper installed between the base and the ground in order to provide controllable damping for the system. They proposed a modified clipped-optimal control strategy by seeking an optimal switching plane (by examining several weightings) for H2/LQG controller. The structural system subjected to a wide range of ground motion intensities and characteristics, and the results of the proposed controller were compared to those where the MR damper was operated in passive mode with a constant current. Significant acceleration reduction over the entire range of considered earthquake intensities was reported. Ramallo et al. [33] compared the performance of a low-damping elastomeric base isolation system equipped with MR dampers, called the smart base isolation system, with several passive base isolation designs using lead-rubber bearings. A clipped-optimal controller was developed for the smart isolation system using an H2/LQG primary controller and a clipping secondary controller to enforce the dissipation requirement. The peak responses of the 2-DOF and 6-DOF models for the base isolated structures subjected to several ground motions were computed from the simulations. According to their conclusions, the proposed smart base isolation system can provide superior protection from a wide range of ground motions, whereas the passive lead-rubber bearing designs tend to be suboptimal for events different from their design earthquake. They showed that due to the adaptive nature of the smart damping system, it can protect the structure against extreme earthquakes without sacrificing performance during the more frequent, moderate seismic events. Nagarajaiah and Narasimhan [60] provided sample active H2/LQG, semi-active clipped optimal and
semi-active skyhook controllers for the three-dimensional base-isolated benchmark problem proposed by Narasimhan et al. [80]. The primary purpose of their study was to illustrate and design active and semi-active controllers so that other researchers can use them as a reference for comparing their controller performance. Guo et al. [100] conducted a shake table testing on a base-isolated highway bridge with MR dampers subjected to earthquake ground motions to evaluate the system performance on the reduction of pounding between adjacent superstructure and base isolation system. MR dampers were installed in series with rubber bearings under the superstructure and the LQR controller was used as the semi-active controller. The results of shaking table tests were also compared with the results from numerical simulations, which showed that the semi-active control system with MR dampers effectively precludes pounding.

2.5.3 Base Isolation Systems Implementing Sliding Mode Control (SMC)

Sliding Mode Control (SMC) is a tool for robust control of nonlinear dynamic systems. SMC requires the construction of a sliding surface on which the error approaches zero which will cause a stable motion on the sliding surface [101, 102]. Fan et al. [78] conducted shake table tests on a three-story steel frame building with a base-isolation system placed on the first floor. The base isolation system consisted of a single 6kN force capacity MR damper together with a sliding friction pendulum isolator and a mass equipment (a piece of mechanical equipment with known mass) to reduce the vibration of the equipment in the first floor. Decentralized Sliding Mode Control (DSMC) and LQG control was used as their semi-active controller, where the command signal was determined only using the local feedback signals. The results of the semi-active controller were compared with passive-on and passive-off cases. Fan et. al concluded that a proper design
of control algorithms for the semi-actively controlled isolation system can reduce the peak response acceleration of the equipment without substantially increasing isolator displacement and building structural response.

2.5.4 Base Isolation Systems Implementing Fuzzy Controllers

Fuzzy logic is one of the main methodologies developed for intelligent control of systems. Fuzzy controllers use expert knowledge instead of the differential equation to describe a system [103]. For the systems that do not have any simple accurate model or nonlinear systems, the fuzzy information can be used to determine desirable control actions through the implementation of rules which relate the input variables to the desired output, or control action.

Jung et al. [65] numerically investigated the effectiveness of four different semi-active control systems for seismic protection of base-isolated building structures implementing MR dampers. They considered the Phase I smart base-isolated benchmark building problem suggested by Narasimhan et al. [80] employing fuzzy logic-based control algorithms, modified clipped-optimal control, maximum energy dissipation, and the modulated homogeneous friction controller. According to the presented results, most of the considered control systems could be beneficial in reducing seismic responses, especially in terms of the base displacement. They showed that if the reduction of the base displacement is of interest with no regard to variation of the floor acceleration, the original clipped optimal control algorithm proposed by Dyke et al. [27] could be recommended. However, if it is necessary to reduce the base displacement without increasing the floor acceleration, their proposed modified clipped-optimal control algorithm could be considered as one of promising candidates for the linear benchmark base-isolated system.
Kim and Roschke [73] used neuro-fuzzy models to represent the dynamic behavior of MR dampers and friction pendulum systems (FPSs) as the base isolation system of a single degree of freedom steel frame. They applied a fuzzy logic controller (FLC) to the MR damper so as to minimize structural acceleration while maintaining acceptable base displacement levels. In order to optimize the parameters of membership functions and find appropriate fuzzy rules, a non-dominated multi-objective genetic algorithm (NSGA-II) was used. The effectiveness of the proposed NSGA-II for FLC was evaluated through numerical simulations under several historical earthquakes. Lin et al. [72] performed experimental studies using shake table testing on a mass equipped with a base-isolation system of high damping rubber bearings (HDRB) and a 300kN MR damper. They proposed three different fuzzy controllers which used feedback displacement, velocity and acceleration data from sensors attached to the structure. Based on the results from various types of passive and semi-active control strategies, they concluded that a combination of HDRB isolators and an adjustable MR Damper can provide robust control of vibration for large civil engineering structures that need protection from near- and far-fault earthquakes. Shook et al. [66] conducted a comparative analytical and experimental study of three different algorithms for the control of seismically excited floor- and base-isolated structures. The isolation layer was compromised of a bidirectional roller-pendulum system (RPS) and MR dampers. The multi-input, single-output neural network control, multi-input, single-output LQR/clipped optimal control with variable gains and multi-input, multi-output fuzzy logic control (FLC) were implemented and the Bouc-Wen model was used to train and predict the behavior of MR dampers. According to their observations, the LQR/clipped optimal controller with variable gains is superior to the other controllers in
50% of the investigated cases, while the fuzzy logic controller performs well for earthquakes with large accelerations. Neural network control is found to be effective in minimizing the acceleration of the superstructure that is subject to moderate excitation. Ali and Ramaswamy [64] developed two optimal FLCs, fixed rule base (FRB) and adaptive rule base (ARB), for the base-isolated nonlinear benchmark building problem proposed by Narasimhan et al. [80]. Acceleration and relative velocity responses at the damper locations were taken as the inputs to the FLC system. They compared the simulation results from their proposed controllers with the sample controller of the benchmark exercise and indicated the improvement provided. Shook et al. [62] evaluated the performance of a proposed hybrid isolation system composed of linear elastomeric bearings (EB), friction-pendulum bearings (FBP), shape memory alloy (SMA) wires and MR dampers with a FLC controller for the benchmark structure proposed by Narasimhan et al. [80]. They employed neuro-fuzzy techniques to model SMA and MR damper elements. Their results showed that the proposed superelastic semi-active base isolation system can reduce the base drift by 18% and maintain a favorable superstructure response. Chen et al. [74] performed real-time hybrid testing on a base-isolated two-degree-of-freedom building. The superstructure and the low-damping base-isolator were numerically simulated and the MR damper was tested physically. The MR damper was controlled by three different control algorithms, including passive-on, LQR and Fuzzy Logic Controller (FLC) in real-time. In order to compensate for the actuator delay and increase the accuracy of the tests, an adaptive phase-lead compensator was used. The accuracy of their tests was evaluated by using the root mean square error and the tracking indicator. Chen et al. concluded, based on the experimental results, that the LQR control and FLC algorithms can effectively reduce the
relative displacements and absolute accelerations of the superstructure compared with the passive-on case.

2.5.5 Base Isolation Systems Implementing Neural Network Controllers

An artificial neural network is another type of intelligent controller. Artificial neural networks were developed for emulating the biology of the human brain, resulting in systems that learn by experience [104]. Due to the nonlinear characteristics of base isolator elements and the MR dampers as well as the nonlinearity of the structure itself, a base-isolated structural system with MR dampers could be a highly nonlinear system. Thus, due to the adaptive and self-organizing performance of neural network controllers, they can be implemented into a complex system if the pre-defined data set for these controllers is well selected. Additionally, the identification of an unknown system and evaluation of its response can be implemented with the neural network method without building a mathematical model of the system [105].

Lee et al. [106] proposed a semi-active neuro-controller for seismic response reduction of the eight-story base-isolated benchmark structure with MR dampers provided by Narasimhan et al. [80]. Their proposed controller adopts a training algorithm based on a cost function and a sensitivity evaluation algorithm to produce the desired control force. They employed a clipped algorithm to induce the MR damper to generate approximately the desired control force by selecting appropriate command voltage. According to their conclusion, the semi-active controller was able to significantly reduce the floor acceleration, base shear, and building corner drift with a slight increase in base displacement. Bani-Hani and Sheban [76] presented and evaluated a semi-active controller-based neural network. The six-story base-isolated building of their study was combined
with MR dampers in the base level. An inverse neural network model (INV-MR) and an
LQG controller were designed to replicate the inverse dynamics of the MR damper and to
produce the optimal control force, respectively. The combination of the INV-MR system
and the LQG controller was implemented for training a semi-active neuro-controller (SA-
NC). The SA-NC controller was used to control the structure by producing the necessary
voltage to MR dampers. They used a passive system of lead-rubber bearings to compare
and assess the performance of the proposed SA-NC system under several historical
earthquake records.

2.5.6 Other Semi-Active Controllers

In addition to the aforementioned semi-active controllers, some other controllers
have been developed by researchers that mostly work based on the feedback data from
different sensors attached to the base-isolated structure. A brief description of these
controllers is provided in this section.

Nitta et al. [68] proposed a new semi-active control strategy for base-isolated
buildings implementing MR dampers. Their proposed scheme controls the magnitude of
the slip-force level of the MR damper based upon the measurement of absolute acceleration
responses, taking into account a simply approximated yet useful relationship between the
supply electric voltage and the set slip-force level. The acceleration responses relative to
the ground at the damper locations and the dampers’ forces were utilized in changing the
slip-force levels. The performance of the proposed system was compared with the clipped
optimal controller for linear isolation systems and skyhook controller for the friction
isolation system proposed by Nagarajaiah and Narasimhan [60, 107, 108]. According to
the results they provided, the proposed controller could reduce the base shear and structure shear without enlarging the base displacements. Chang et al. [61] presented a semi-active control strategy for the benchmarked base-isolated structure proposed by Narasimhan et al. [80] by implementing combined linear and nonlinear isolators with the aim of reducing the displacement at the base level. In order to reduce the computational resources, they used a reduced-order model for structural behavior and employed an optimal direct output feedback control algorithm combined with nonlinear models. As shown in their results, the control strategies can mitigate displacement response at the base significantly while acceleration response is slightly increased at the same time by employing the proposed MR dampers or the MR dampers of the benchmark control problem. Choi et al. [63] numerically investigated the applicability of the MR damper-based smart passive control with electromagnetic induction (EMI) part for seismic protection of benchmark base-isolated building proposed by Narasimhan et al. [80] with nonlinear isolation elements such as FPBs and LRBs. The EMI part consisted of a permanent magnet and a solenoid coil which was used to produce electric energy to the damper by changing the kinetic energy of the reciprocation motion of the MR damper to electric energy, instead of using an external power supply. Base on their conclusion, the proposed system had a comparable and superior control performance to the conventional MR damper-based semi-active control systems of Lyapunov control algorithm, maximum energy dissipation algorithm, modulated homogeneous friction algorithm and intelligent neural network algorithm. Tu et al. [38] implemented a generalized substructuring framework to evaluate the performance of a single-axis base-isolated structure, dynamically substructured system (IS-DSS) equipped with an MR damper. The MR damper was considered as the physical
substructure and the superstructure and the roller-pendulum systems constituted the linear numerical substructure. A linear inverse dynamics compensation via simulation (IDCS) and adaptive minimal control synthetic algorithm with error feedback (MCSEF) controllers were tailored for the control of the IS-DSS. The effectiveness of the adaptive substructuring method against conventional shake-table testing was compared, and a 1.32% error was reported. They concluded that the accuracy of the substructuring method compared with the response of the shaking-table is dependent upon the fidelity of the numerical substructure.

2.6 Summary

In this chapter, accurate MR damper and base isolator models and different semi-active control strategies for base-isolated structures with MR dampers have been reviewed. It is essential to have accurate models for MR dampers and base isolators to be able to numerically model the isolation layer and evaluate the efficiency of the control algorithms as well as the performance of the structural system subjected to earthquake ground motions. Although various kinds of semi-active control algorithms have been developed, a majority of the studies have focused on the effectiveness of the algorithm without providing practical design procedures for incorporating these control laws into real structures. Without any systematic design procedure, designers have no idea how many and what type of semi-active devices (e.g. MR dampers) need to be used to satisfy the given performance objectives to their structures. Furthermore, the existing methods have not been rigorously validated under earthquake ground motions with various frequency contents (i.e. long- or short-period motions). Additionally, due to the huge cost of experimental studies, most of these studies were conducted numerically, and the implementation of experimental
validations were quite limited. Moreover, studies comparing the cost for implementing the passive and semi-active controllers were not sufficiently conducted yet.
CHAPTER 3

TRANSMISSIBILITY-BASED SEMI-ACTIVE (TSA) CONTROL ALGORITHM

3.1 General

In this chapter, a new adaptive control algorithm for base-isolated structures with semi-active damping devices is introduced which covers the limitations of the existing controllers described in Chapter 2. Due to the shortcomings, primarily related to the lack of knowledge for quantitatively predicting the response of the semi-actively controlled base isolation system under earthquake loads, the practical application of semi-actively controlled base isolation systems is quite limited. Actually, existing control laws for the semi-active devices are often based on control objectives that are not directly related to minimizing the maximum structural responses of interest (e.g., the isolator displacement, story drifts, floor accelerations) [40]. For instance, the performance of controllers such as the linear quadratic regulator (LQR) and the sliding mode control (SMC) depends on the proper selection of the weighting matrices along with the gradient vector of the sliding surface [29], respectively, which is actually a challenging task for the design of semi-active controllers for real structures. Additionally, the control objective in the design of these controllers is based on minimizing a quadratic cost function over the entire control time [99]. Chae [29] and chae et al. [2, 95] showed that minimizing the quadratic cost function does not always lead to the minimization of the maximum structural response. Intelligent controllers such as fuzzy control and neural networks work based on developing a nonlinear system that correlates the input (e.g., feedback response) and output (e.g., command signal to semi-active devices) data. The optimization of the nonlinear system depends on the training set (for neural networks) and fuzzy logics (fuzzy controls) tuned
from selected ground motions. Thus, the performance of these controllers depends on the selection of a pre-defined data set, implying the controller may not work well for other inputs that have different characteristics from the pre-defined data set. In addition, the quantitative prediction of expected structural responses using these controllers is also difficult due to the complexity of the nonlinear system. Therefore, it is not easy to estimate the response of structures with these control laws, which makes it difficult to incorporate these control laws into practical design methods. The *Transmissibility-Based Semi-Active* (TSA) control law introduced in this chapter can adaptively change the damping characteristics based on the period contents in the response of structures. Basically, the TSA controller is intended to maximize the damping of the isolation system under long-period ground motions, while it minimizes the damping under short-period ground motions. Furthermore, unlike existing methods, the TSA control law enables the response prediction of base-isolated structures with semi-active damping devices without implementing any complex nonlinear time history analysis, which can be incorporated into a practical design procedure. Another unique feature of the proposed system is that the proposed controller only requires local feedback data. Most controllers in the existing studies require full-state feedback data of a structure, making the feedback system for the controller expensive. However, the proposed controller only requires acceleration at the base floor, which will simplify the feedback system and reduce costs.

### 3.2 Transmissibility-Based Semi-Active (TSA) Control Law

Once a base-isolated structure is subjected to a long-period earthquake ground motion, the response of the structure can be amplified by the resonance effect. It is well-known that the resonance issue can be effectively resolved by increasing the system
damping. However, the use of large damping is not beneficial under a short-period earthquake ground motion. These characteristics of damping on the performance of base isolation system is well-demonstrated through the transmissibility theory shown in Figure 3.1, where the transmissibility ratios of a single-degree-of-freedom (SDOF) structure are compared for the cases with low damping ($\zeta = 5\%$) and high damping ($\zeta = 30\%$). Since the motion of a base-isolated structure is predominantly governed by its fundamental mode, the entire base-isolated structure can be approximated as an SDOF structure. As can be observed in Figure 3.1, high damping is beneficial to reduce the transmitted force to the superstructure when the frequency ratio $f/f_n$ is less than $\sqrt{2}$ (i.e., long-period excitation), where $f$ and $f_n$ are the excitation frequency and the natural frequency of the structure, respectively. In particular, the use of high damping is very effective to reduce the transmitted force under the resonance state, which will significantly reduce the isolator displacement as well as other structural responses. When $f/f_n$ is greater than $\sqrt{2}$ (i.e., short-period excitation), however, high damping increases the transmitted force, which would be harmful to the superstructure. Therefore, the use of low damping is desirable in this case.
The proposed control law is based on the observation from the transmissibility theory, where the damping force is controlled to be maximum under long-period excitations, while to be minimum under short-period excitations, in order to effectively reduce the transmitted force to the superstructure over a wide excitation frequency ranges. This control law is similar to the one suggested by Chae and Ricles [40], but the proposed transmissibility-based semi-active (TSA) control law is modified to improve the control performance further. The damper force $f_d$ is determined as follows in the proposed TSA control law [8]:

$$
\text{Damper command} = \begin{cases} 
\text{Activation} & (f_d = f_{d,\text{max}}) ; \quad \text{if } T_c \geq \alpha_1 T_{cr} \\
\text{Deactivation} & (f_d = f_{d,\text{min}}) ; \quad \text{if } T_c \leq \alpha_2 T_{cr}
\end{cases} \tag{3.1}
$$
where, \( f_{d,\text{max}} \) and \( f_{d,\text{min}} \) are the maximum and minimum damper force, respectively, which can be achieved by controlling the command signal to the damper (e.g., in case of MR dampers, \( f_{d,\text{max}} \) and \( f_{d,\text{min}} \) are the damper forces when the input currents are maximum and minimum, respectively). \( T_{cr} \) is the critical period which is defined as: 

\[
T_{cr} = \left( \frac{1}{\sqrt{2}} \right) T_{iso},
\]

where \( T_{iso} \) is the fundamental period of the isolated structure. \( \alpha_1 \) and \( \alpha_2 \) are user-defined parameters, which are less than or equal to 1.0. \( T_c \) is the characteristic period that is determined based on the time between zero-crossings of the base floor acceleration \( a_{base} \) (see Figure 3.2). Although the earthquake response of a structure is quite different from the steady-state response of the structure under harmonic excitation, \( a_{base} \) can provide a useful information for excitation frequency that can be applied toward the transmissibility theory in Figure 3.1. For example, period contents that are shorter than \( T_{iso} \) will be predominantly observed in \( a_{base} \) if the base-isolated structure is subjected to a short-period earthquake ground motion. On the other hand, if the base-isolated structure is subjected to a long-period ground motion, period contents that are close to \( T_{iso} \) will be observed more in \( a_{base} \), where the damper may need to be activated since the use of high damping is effective in such a case. At every time step, \( T_c \) is assigned to be the same as the previous time step’s \( T_c \) unless zero-crossing occurs; if zero-crossing occurs, \( T_c \) at that time step will be updated as described in Figure 3.2. \( T_c \) is set to be zero initially.
For a harmonic excitation leading to a steady-state response, the theoretical value of $\alpha_1$ is 1.0, and the damper will be activated when $T_c \geq T_{cr}$. However, the situation will be different under earthquake loads. Natural earthquake ground motions contain a wide range of frequency contents. Even a strong long-period ground motion includes various degrees of high-frequency contents, resulting in small $T_c$ values that make the damper activation delayed or impossible. Therefore, $\alpha_1$ needs to be less than 1.0 under earthquake loads.

Once the damper is activated, it is deactivated only when the deactivation condition given in equation (3.1) is satisfied. Based on Figure 3.1, a harmonic excitation can be classified as a short-period excitation if $T_c$ is less than $T_{cr} (=T_{iso}/\sqrt{2})$, where the damper needs to be deactivated. Under earthquake loading, $T_c$ can be frequently less than $T_{cr}$ even under a strong long-period earthquake ground motion as explained earlier. Keeping the damper activated will make the system effective under such a strong long-period earthquake ground motion, rather than having the damper frequently change its states by the inherent high-frequency contents in the ground motion. The deactivation condition in
equation (3.1) was developed to effectively avoid this problem, where \( \bar{T}_c \) is the average of recent \( T_c \) values over the time length of the grant time \( t_{GT} \), and \( \alpha_2 \) is a constant less than 1. By having the average value of \( T_c \), the deactivation will be less affected by the high-frequency excitation. Furthermore, once the damper is activated, the damper is controlled to be in the activation mode at least for the duration of \( t_{GT} \) in the proposed TSA control law. That is to say, the damper is not deactivated during the initial \( t_{GT} \) period, and the average process of \( T_c \) to check the deactivation condition is only implemented after \( t_{GT} \) from the moment of activation. This will also make the deactivation decision less affected by disturbance from the high-frequency excitation. Once the damper is deactivated, it will remain deactivated until the activation condition is met.

\[ t_{GT}, \alpha_1, \text{ and } \alpha_2 \text{ are user-defined parameters. If } t_{GT} \text{ is set to be long and a strong short-period ground motion is assumed to be followed by a long-period motion, the damper can still remain activated even under the strong short-period motion, which is not desirable. If } t_{GT} \text{ is too small, deactivation can be affected by the high-frequency contents in the ground motion. Therefore, appropriate values for these parameters need to be used. In order to further resolve the high-frequency excitation issue, it is strongly suggested to apply a low-pass filter to } a_{base} \text{ before finding } T_c. \] The use of a low-pass filter will also effectively resolve the noise issue in the measured acceleration signal of \( a_{base} \).

It is noteworthy that the proposed TSA controller is a decentralized one that only needs local feedback data, \( a_{base} \), while most existing semi-active control algorithms for base isolation systems require full-state feedback data. Thus, the cost for constructing a sensor network for signal feedback can be significantly reduced by using the proposed TSA control law.
3.3 Summary

In this chapter, a new transmissibility-based semi-active (TSA) control law was developed in order to improve the performance of base-isolated structures with semi-active damping devices under earthquake ground motions with various frequency contents. The control law is based on the transmissibility theory of an SDOF structure subjected to a steady-state response under harmonic loading, where the use of high and low damping are beneficial under long- and short-period excitations, respectively. Based on the TSA control law, the damping of the base-isolated structure can be adaptively changed depending on the frequency contents in the response of the base-isolated structure subjected to earthquake loading to achieve maximum damping under long-period excitations and minimum damping under short-period excitations. The TSA controller is also cost-effective since it does not require the full-state feedback of the structural response, unlike most existing controllers. Furthermore, it can be incorporated into a practical design procedure, which is called the Simplified Design Procedure in this dissertation and will be discussed in detail in Chapter 4.
CHAPTER 4
SIMPLIFIED DESIGN PROCEDURE FOR BASE-ISOLATED STRUCTURES
WITH MR DAMPERS

4.1 General

In order to practically implement semi-active damping devices into the structures, systematic design procedures should be developed and used. Without any systematic design procedure, designers have no idea how many and what type of semi-active devices need to be used to satisfy the given performance objectives to their structures to be designed. In order to improve the shortcomings related to the practical design of semi-active systems, Fan [109] evaluated the performance of non-ductile reinforced concrete frame buildings with viscoelastic dampers and proposed a design procedure for a structure with viscoelastic dampers. Lee et al. [110, 111] presented a systematic design procedure for the preliminary seismic design of frame buildings with viscoelastic or high-damping elastomeric materials. They conducted numerical nonlinear dynamic time history analyses to show that the steel special moment resistant frames (SMRFs) designed based on their proposed design procedure can achieve the specified seismic performance objectives. Chae [29] developed a systematic analysis procedure for the design of single-degree-of-freedom structures with MR dampers by implementing a quasi-static MR damper model for determining the loss factor and effective stiffness. By extending the proposed simplified analysis procedure, Chae [29] also formulated a design procedure for multi-degree-of-freedom structures with MR dampers. In order to evaluate the performance of the proposed procedure, the expected response was compared with a series of nonlinear time history analyses using OpenSees [112]. He proved that the Herschel-Bulkley quasi-static model
for estimating the effective stiffness and energy dissipation for MR dampers can satisfactorily estimate the response of structures with MR dampers under various earthquake loads.

As explained in Chapter 3, the proposed TSA control law maximizes the damping force under long-period ground motions, while it minimizes the damping force under short-period ground motions. This control law can be achieved as well by operating the base-isolated structure with its maximum damping (i.e., activation case in equation (3.1)) under normal conditions and deactivating the damper only when the deactivation condition is satisfied. Therefore, the base isolated structure with the proposed TSA control law can be designed first as a typical passive base isolation system, where the semi-active damping device is passively controlled with its maximum input command, i.e. passive-on system. If the semi-active damping device is passively controlled with its minimum input command, it is called the passive-off isolation system in this dissertation. It should be mentioned that the TSA control algorithm is applicable for base-isolated structures in combination with any kind of semi-active damping devices able to switch between the maximum and minimum damping modes.

The proposed systematic design procedure of this study is called the Simplified Design Procedure which is able to predict the behavior of base-isolated structures with MR dampers. This procedure combines the bilinear model for base isolators and the quasi-static Herschel-Bulkley model for MR dampers.

4.2 Simplified Design Procedure

In this section, first, details related to energy dissipation and effective stiffness of the bilinear model for base isolators are provided. Then, different quasi-static models for
MR dampers will be presented, and the procedure for identifying the effective stiffness and equivalent damping ratio based on the Herschel-Bulkley quasi-static model will be introduced. Lastly, details of the combined models of the bilinear base isolator model and the Herschel-Bulkley model will be provided, along with the procedure how to estimate the response of the base isolation system with MR dampers.

### 4.2.1 Bilinear Model for Base Isolators

The effective stiffness and the amount of dissipated energy of base isolators can be reasonably estimated by using a bilinear model. Figure 4.1 shows a bilinear model for base isolators. Prior to the yielding of the isolator, the initial stiffness of the isolator is found from the ratio of the force to the corresponding displacement (i.e., $k_i$). After the yielding point, (where $Q_y$ and $x_y$ are the yield force and yield displacement, respectively), the relationship between the force and displacement is correlated with the yield stiffness, $k_y$. At the maximum displacement of $x_0$, the corresponding restoring force from the base isolator is maximum as well (i.e., $f_{max,BI}$). Once the motion is reversed, the force will decrease along the line with the same slope as the initial stiffness. Then, the force-displacement relation continues with the slope of $k_y$, once the isolator has yielding again at the negative force side, up to the maximum negative displacement.
The maximum restoring force from the isolator, $f_{\text{max,BI}}$, is of interest in the simplified design procedure which equals

$$f_{\text{max,BI}} = Q_y + k_y (x_0 - x_y)$$  \hspace{1cm} (4.1)

### 4.2.1.1 Effective Stiffness of Bilinear Model

Due to the nonlinear performance of the base isolators, the effective stiffness should be defined accurately so that the natural period of the structure and the corresponding response of the base-isolated structure can be well-estimated. The effective stiffness is defined herein as the ratio of the maximum force corresponding to the maximum displacement:

$$k_{\text{eff,BI}} = f_{\text{max,BI}} / x_0$$ \hspace{1cm} (4.2)

where, $f_{\text{max,BI}}$ is the corresponding restoring force of base isolator at the maximum displacement of $x_0$. The definition of $k_{\text{eff,BI}}$ for the bilinear model of base isolators is schematically shown in Figure 4.1.
4.2.1.2 Energy Dissipation of Bilinear Model

The amount of energy dissipation by the bilinear model is determined from the area of the hysteresis loop of the force-displacement relationship, which is given in the following equation.

\[ E_{D,BI} = 4 \left( x_0 Q_y - x_y f_{\text{max, BI}} \right) \]  \hspace{1cm} (4.3)

4.2.2 Quasi-Static Models for MR Dampers

Three different quasi-static models which can be used for modeling the response of an MR damper are provided in this chapter, i.e., the simple frictional model, the Bingham model, and the Herschel-Bulkley models [29]. The MR damper force is dependent on the velocity. Once the damper is subjected to a constant velocity, the quasi-static behavior is observed. The simple frictional model has a rectangular shape force-displacement loop as shown in Figure 4.2, and the damper force is presented as

\[ f_{MR} = f_0 \text{sign}(\dot{x}) \]  \hspace{1cm} (4.4)

\(x\) and \(\dot{x}\) in equation (4.4) are the damper displacement and velocity, respectively. \text{sign()} is the signum function; if \(\dot{x}\) is positive, \text{sign}(\dot{x}) is equal to positive one, otherwise, it is negative one.
Figure 4.2 Simple frictional model for MR dampers; (a) force-displacement relationship, (b) force-velocity relationship

The Bingham quasi-static MR damper model consists of a frictional element in parallel with a linear dashpot. The force-displacement relationship of the Bingham model is shown in Figure 4.3, where the damper force is given as

\[ f_{MR} = f_0 \text{sign}(\dot{x}) + C\dot{x} \]  \hspace{1cm} (4.5)

\( C \) in equation (4.5) is the dashpot coefficient of the Bingham model.
The quasi-static Herschel-Bulkley model for MR dampers consists of a nonlinear viscous dashpot in parallel with a frictional element. Figure 4.4 shows the force-displacement relationship of this model. The damper force is given as

$$f_{MR} = (f_0 + C|\dot{x}_0|^n)\text{sign}(\dot{x})$$  \hspace{1cm} (4.6)$$

By changing the Herschel-Bulkley parameters of \(C\) and \(n\), it can model both the Bingham model, with \(n=1\), and the simple friction model, with \(C=0\). Depending on the input current to the MR damper, the values for \(C\), \(n\) and \(f_0\) parameters can be appropriately defined by conducting characterization tests. Based on equation (4.6), the maximum damper force of the Herschel-Bulkley model is

$$f_{max, MR} = f_0 + C|\dot{x}_0|^n$$  \hspace{1cm} (4.7)$$

The accuracy of the Herschel-Bulkley quasi-static model for estimating the response of the structural systems with MR dampers has been demonstrated by Chae and Chae et al. [29, 113].
4.2.2.1 Effective Stiffness of the Herschel-Bulkley Quasi-Static Model

Depending on the amount of current input to the MR damper, the stiffness provided by the damper to the structure will vary. Thus, the effective stiffness of the MR damper has a direct influence on the natural period of the structure and should be well-defined accordingly. The same procedure used in defining the effective stiffness of the bilinear model for base isolators is used for MR dampers in this section. From Figure 4.4, the effective stiffness of a passively-controlled MR damper is defined to be:

\[ k_{\text{eff,MR}} = \frac{f_0}{x_0} \]  

(4.8)

where, \( f_0 \) is the corresponding damper force at the maximum displacement of \( x_0 \).

4.2.2.2 Energy Dissipation of the Herschel-Bulkley Quasi-Static Model

Assuming that a harmonic displacement motion of \( x(t) \) with the amplitude of \( x_0 \) and the excitation frequency of \( \omega \) is applied to the MR damper as

\[ x(t) = x_0 \sin(\omega t) \]  

(4.9)
then, the dissipated energy in one cycle of the harmonic motion will be equal to

\[ E_{D,MR} = \int_0^{\frac{2\pi}{\omega}} f(t)\dot{x}(t)\,dt \quad (4.10) \]

Substituting equations (4.6) and (4.9) into equation (4.10), and then evaluating the integration of equation (4.10) will result in the following equation for the amount of dissipated energy of the Herschell-Bulkley model over a complete single cycle:

\[ E_{D,MR} = 4f_0x_0 + 2^{n+2}\zeta\gamma(n)x_0^{1+n}\omega^n \quad (4.11) \]

where

\[ \gamma(n) = \frac{\Gamma^2\left(1 + \frac{n}{2}\right)}{\Gamma(2 + n)} \quad (4.12) \]

and \(\Gamma()\) is the gamma function [24]. When \(n=1\), the Herschell-Bulkley model becomes the Bingham model, and the amount of dissipated energy for the Bingham model is

\[ E_{D,MR} = 4f_0x_0 + \pi C\omega x_0^2 \quad (4.13) \]

The amount of dissipated energy of the simple frictional is obtained by inserting \(C=0\) into equation (4.13)

\[ E_{D,MR} = 4f_0x_0 \quad (4.14) \]

### 4.2.3 Combined System of Base Isolators and MR Dampers

As long as the effective period \((T_{eff})\) and the equivalent damping ratio \((\zeta_{eq})\) for the passive-on isolation system are identified, the passive-on isolation system can be designed by incorporating \(T_{eff}\) and \(\zeta_{eq}\) into the current seismic design procedure for base-isolated structures such as provided in ASCE7 (2010) [1]. Then, the base isolation system can be controlled with the proposed TSA control law to maximize its effectiveness under both long- and short-period earthquake ground motions.
$T_{\text{eff}}$ and $\zeta_{\text{eq}}$ for an isolation system can be identified from the typical force-deformation relationships of base isolators and passively controlled semi-active dampers. It should be noted that the motion of a base-isolated structure is predominantly governed by its fundamental mode. Thus, the entire base-isolated structure can be approximated as an SDOF structure.

The goal here is to find the maximum displacement $x_0$ and the maximum base shear $V_{\text{max}}$ of an isolation system using the design spectrum in terms of the given properties for the base isolator and MR damper. In order to identify the effective period of the base isolation system, the effective stiffness of the base isolator ($k_{\text{eff},BI}$) and MR damper ($k_{\text{eff},MR}$) explained in detail in previous sections should be employed. Since the isolator and MR damper are installed in parallel, they will have the same maximum displacement $x_0$. Thus, the effective period of the isolation system is determined as

$$T_{\text{eff}} = 2\pi \sqrt{m_t / (k_{\text{eff},BI} + k_{\text{eff},MR})}$$

(4.15)

where, $m_t$ is the total mass of the isolated building. It should be noted that $T_{\text{eff}}$ is a function of $x_0$. The equivalent damping ratio $\zeta_{\text{eq}}$ of the isolation system is determined from the strain energy and dissipated energy over a single cycle. The strain energy of the system $E_S$ is

$$E_S = \frac{1}{2} (k_{\text{eff},BI} + k_{\text{eff},MR}) x_0^2$$

(4.16)

The dissipated energy over a single cycle $E_D$ is the same as the sum of the dissipated energy by the base isolator and the MR damper from their hysteresis loops as

$$E_D = E_{D, BI} + E_{D, MR}$$

(4.17)

Then, the equivalent damping ratio of the isolation system can be found as
\[ \zeta_{eq} = \frac{1}{4\pi} \frac{E_D}{E_s} \quad (4.18) \]

With the identified \( T_{eff} \) and \( \zeta_{eq} \) from equations (4.15) and (4.18), \( x_0 \) can be found using the design spectrum for displacement, \( S_D \), as

\[ x_0 = S_D(T_{eff}, \zeta_{eq}) \quad (4.19) \]

It should be noted that \( T_{eff} \) and \( \zeta_{eq} \) are nonlinear functions of \( x_0 \), making it difficult to analytically solve equation (4.19) for \( x_0 \). Thus, equation (4.19) can be solved iteratively with an assumed initial value of \( x_0 \). For the given \( x_0 \), \( T_{eff} \) and \( \zeta_{eq} \) are updated at each iteration step using equations (4.15) and (4.18), respectively. It should be also noted that ASCE7-10 [1] provides coefficients (i.e., \( B_D \) and \( B_M \) for the design based and maximum considered earthquake ground motions, respectively) for various equivalent damping ratios \( \zeta_{eq} \) which should be considered at each iteration step for finding \( x_0 \). Then, new \( x_0 \) is obtained from equation (4.19), and this procedure needs to be repeated until \( x_0 \) converges.

Once \( x_0 \) is found from the iteration procedure, this can be used to design the maximum displacement of the base isolators and MR dampers. The maximum design force developed in the isolator can be found from equation (4.1) and the maximum design force for the MR damper is determined from equation (4.7). It should be noted that the maximum design velocity of the damper \( \dot{x}_o \) is obtained from the pseudo-velocity as follows

\[ \dot{x}_0 = \left( \frac{2\pi}{T_{eff}} \right) x_0 \quad (4.20) \]

The maximum design shear force \( V_{max} \) transmitted to the superstructure by the base isolator and MR damper is equal to the sum of the maximum forces from equations (4.1) and (4.7) as

\[ V_{max} = f_{max, B1} + f_{max, MR} \quad (4.21) \]
which can be used for designing the superstructure. The block diagram providing the
details of the simplified design procedure is shown in Figure 4.5.
Figure 4.5 Block diagram of the simplified design procedure for base-isolated structures with MR dampers

Assume \( x_0 \) and set \( T_{\text{eff}} = \frac{2\pi}{\omega} \sqrt{m_c / k_{\text{eff}, BI} + k_{\text{eff}, MR}} \)

Where, \( k_{\text{eff}, BI} = f_{\text{max}, BI} / x_0 \) and \( k_{\text{eff}, MR} = f_0 / \xi_0 \)

\( f_{\text{max}, BI} = Q_y + k_y (x_0 - x_y) \) and \( f_{\text{max}, MR} = f_0 + C |\xi_0|^n \)

Determine the Dissipated Energy by the Base Isolator, \( E_{D, BI} \)

\( = \) Hysteretic Area of the model, \( E_{D, BI} = 4(x_0 Q_y - x_y f_{\text{max}, BI}) \)

Determine the Dissipated Energy by the MR Damper, \( E_{D, MR} \)

\( E_{D, MR} = 4 f_0 x_0 + \pi C \xi_0^2 \)

Calculate the whole Dissipated Energy by the Isolation System

\( E_D = E_{D, BI} + E_{D, MR} \)

Calculate the Strain Energy of the Dynamic System

\( E_s = \frac{1}{2} (k_{\text{eff}, BI} + k_{\text{eff}, MR}) x_0^2 \)

Calculate the Equivalent Damping Ratio of the Isolation System

\( \xi_{eq} = \frac{1}{4 \pi} \frac{E_D}{E_s} \)

Find the Maximum Displacement from the Response Spectrum

\( x_0 = S_D(T_{\text{eff}}, \xi_{eq}) \)

\( \text{if} \quad |x_0^{new} - x_0| > \text{Tolerance?} \)

\( \text{Yes} \quad \text{Set} \quad x_0 = x_0^{new} \)

\( \text{No} \)

Set \( x_0 = x_0^{new} \) and Determine the Maximum Design Force for MR Damper, and Maximum Force Developed in the Base Isolator

\( f_{\text{max}, MR} = f_0 + C |\xi_0|^n = f_0 + C (x_0 + \frac{2\pi}{T_{\text{eff}}})^n \)

\( f_{\text{max}, BI} = Q_y + k_y (x_0 - x_y) \)

Determine the Maximum Shear Force Transmitted to the Superstructure

\( V_{\text{max}} = f_{\text{max}, BI} + f_{\text{max}, MR} \)
4.3 Summary

In this chapter, a systematic procedure for estimating the response of the base-isolated structures with MR dampers was developed based on the abilities provided by the TSA control law. This procedure was named the simplified design procedure, enabling the design of base-isolated structures with MR dampers without conducting nonlinear time history analysis. The simplified design procedure works based on the effective period and equivalent damping ratio. The equations describing the energy dissipation and effective stiffness of the base isolators and MR dampers were presented using the bilinear model and the Herschel-Bulkley model, respectively.

The performance of the TSA controller and the accuracy of the simplified design procedure will be evaluated numerically in Chapter 5 for a full-scale base-isolated three-story building with MR dampers. Experimental evaluation of the proposed methods will be presented in Chapter 6 through conducting real-time hybrid simulations for a small-scale base-isolated structure with MR dampers.
CHAPTER 5
NUMERICAL ASSESSMENT OF THE ADAPTIVE BASE ISOLATION

5.1 General

In this chapter, the performance of the TSA control system proposed in Chapter 3 is assessed by conducting numerical simulations. The simplified design procedure developed in Chapter 4 is incorporated into the design of a three-story base-isolated building with MR dampers. The highly nonlinear behavior of the base isolated system is analyzed by using the Maxwell Nonlinear Slider (MNS) model and the Bouc-Wen model for the MR dampers and the base isolators, respectively, into the numerical simulations based on the nonlinear time history analysis (NTHA). Various earthquake ground motions are employed for the NTHA in this Chapter. The earthquake ground motions are classified into long-period and short-period motions based on the frequency contents and the natural periods of the base isolated structures. The responses of the structure under these earthquake ground motions are provided along with the statistical analysis of the results.

5.2 Design of a Base-Isolated Building with MR Dampers

A base-isolated three-story building with MR dampers shown in Figure 5.1 is designed in this section by using the simplified design procedure to validate the performance of the proposed TSA controller. The building is assumed to be located in the Los Angeles area, California, where the design spectrum of the building is given in accordance with ASCE7-10 [1]. The spectral accelerations for short period and 1-second period are taken as 2.0g and 1.0g, respectively. The site is assumed to be a class B area (Site Class B). For simplicity, the superstructure is assumed as a shear building with floor
masses and story stiffnesses as: \( m_b = m_1 = m_2 = m_3 = 150 \text{ kN} \cdot \text{sec}^2/\text{m} \), \( k_1 = 33,264 \text{ kN/m} \), \( k_2 = 25,588 \text{ kN/m} \) and \( k_3 = 17,912 \text{ kN/m} \).

The design parameters for the base isolator and MR damper are provided in Tables 5.1 and 5.2, respectively. A detailed explanation of the parameters in these two tables is provided in Chapter 2. When the MR damper is controlled passively with its maximum input current (i.e., \( I = I_{\text{max}} \)), the system becomes the passive-on isolation system. On the other hand, it becomes the passive-off isolation system when the input current to the MR damper is zero (i.e., \( I = 0 \)). With these design parameters, the passive-off and passive-on isolation systems are designed by using the simplified design procedure provided in Chapter 4, where the design results of each structural system are provided in Table 5.3. It
can be shown that the equivalent damping ratios for the passive-off and passive-on isolation systems are $\zeta_{eq} = 10\%$ and $\zeta_{eq} = 40\%$, respectively, implying that the system damping can be changed between these two values by controlling the input current to the MR damper of this study.

<table>
<thead>
<tr>
<th>Table 5.1 Design parameters for the base isolator</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bilinear model parameters</strong></td>
</tr>
<tr>
<td>$Q_y$ (kN)</td>
</tr>
<tr>
<td>-----------</td>
</tr>
<tr>
<td>100.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 5.2 Design parameters for the MR damper</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Hershel-Bulkley MR damper model</strong></td>
</tr>
<tr>
<td>Input current</td>
</tr>
<tr>
<td>-------------</td>
</tr>
<tr>
<td>$I = I_{max}$ (passive-on)</td>
</tr>
<tr>
<td>$I = 0$ (passive-off)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 5.3 Design results of each structural system</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without base isolation (fixed-base)</td>
</tr>
<tr>
<td>Natural periods (s)</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Maximum isolation displacement ($u_o$, m)</td>
</tr>
<tr>
<td>Equivalent damping ratio ($\zeta_{eq}$, %)</td>
</tr>
<tr>
<td>Ratio of base shear to total weight ($V_{max}$/$W$)</td>
</tr>
</tbody>
</table>

The values provided in Table 5.3 are obtained from the design basis earthquake (DBE) level design spectrum. With the use of isolators, the fundamental periods of the base
isolated structures are elongated from 1.04s to 2.75s and 2.59s for the passive-off and passive-on isolation systems, respectively. The natural periods of the passive-off and passive-on isolation systems were calculated by incorporating the identified effective stiffness of the base isolator and MR damper from the simplified design procedure (i.e., equations (4.2) and (4.8), respectively) into the stiffness matrix of the entire system. The effective periods determined from the simplified design procedure are 2.61s and 2.44s for the passive-off and passive-on isolation systems, respectively, which are closely made to the fundamental periods of the passive-off and passive-on isolation systems for the three-story building. The fundamental period of the passive-off isolation system is larger than that of the passive-on isolation system because the larger MR damper force of the passive-on isolation system increases the effective stiffness of the entire system. The larger damper force also resulted in an increased effective damping ratio, which can significantly reduce the base isolator displacement by dissipating more energy. The base shear is significantly reduced by having an isolation system, as can be observed from the ratios of the base shear to the total building weight ($V_{max}/W$) in Table 5.3, showing the well-known advantage of the use of the base isolation system. However, no significant difference is observed in the ratios of $V_{max}/W$ between the passive-off and passive-on isolation systems. The increased MR damper force in the passive-on isolation system can increase the base shear, but the increased damping ratio can also effectively reduce the spectral acceleration as well as the base shear; thus, the base shear from the passive-on isolation system can be similar to that from the passive-off isolation system as provided in Table 5.3.
5.3 Selected Earthquake Ground Motions

The designed superstructure is subjected to various earthquake ground motions provided in this section. A given earthquake ground motion can be considered as a short-period or long-period motion depending on the fundamental period of a structure and the predominant frequency contents in the earthquake. If the ratio of the spectral acceleration of a given earthquake at the fundamental period of the passive-on isolation system to that at the fundamental period of the structure without base isolation is greater than 0.4, the earthquake ground motion is classified as a long-period motion; otherwise, it is classified as a short-period motion. Tables 5.4 and 5.5 provide the list of selected long- and short-period earthquake ground motions based on this classification rule.

<table>
<thead>
<tr>
<th>ID</th>
<th>Earthquake</th>
<th>Year</th>
<th>Station</th>
<th>Magnitude</th>
<th>Component</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Imperial Valley</td>
<td>1979</td>
<td>El Centro Array No. 6</td>
<td>6.5</td>
<td>230</td>
</tr>
<tr>
<td>2</td>
<td>Northridge</td>
<td>1994</td>
<td>Sylmar Converter</td>
<td>6.7</td>
<td>052</td>
</tr>
<tr>
<td>3</td>
<td>Manjil, Iran</td>
<td>1990</td>
<td>Abbar</td>
<td>7.4</td>
<td>Transverse</td>
</tr>
<tr>
<td>4</td>
<td>Northridge</td>
<td>1994</td>
<td>Jensen</td>
<td>6.7</td>
<td>022</td>
</tr>
<tr>
<td>5</td>
<td>Chi-Chi, Taiwan</td>
<td>1999</td>
<td>CHY101</td>
<td>6.3</td>
<td>E</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ID</th>
<th>Earthquake</th>
<th>Year</th>
<th>Station</th>
<th>Magnitude</th>
<th>Component</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Northridge</td>
<td>1994</td>
<td>Canyon County-WLC</td>
<td>6.7</td>
<td>270</td>
</tr>
<tr>
<td>2</td>
<td>Landers</td>
<td>1992</td>
<td>Coolwater</td>
<td>7.3</td>
<td>Longitudinal</td>
</tr>
<tr>
<td>3</td>
<td>Loma Prieta</td>
<td>1989</td>
<td>Capitola CDMG 47125</td>
<td>6.9</td>
<td>090</td>
</tr>
<tr>
<td>4</td>
<td>Northridge</td>
<td>1994</td>
<td>Beverly Hills</td>
<td>5.3</td>
<td>279</td>
</tr>
<tr>
<td>5</td>
<td>Kocaeli, Turkey</td>
<td>1999</td>
<td>Duzce</td>
<td>7.5</td>
<td>180</td>
</tr>
</tbody>
</table>
Spectral accelerations of these long- and short-period earthquake ground motions are plotted in Figure 5.2 along with their average values. It can be clearly observed that the averaged spectral accelerations of the long-period earthquake ground motions are much larger than those of the short-period earthquake ground motions around the fundamental periods of the isolation systems.

Figure 5.2 Spectral accelerations for selected long- and short-period earthquake ground motions

5.4 Numerical Modeling of the Three-Story Base-Isolated Building

The equations of motion of the base-isolated three-story building with MR dampers in Figure 5.1 is given as:

\[ M\ddot{u} + C\dot{u} + Ku = \Lambda f - M\Gamma \ddot{x}_g \]  

(5.1)
In equation (5.1), $\mathbf{M}$ is the diagonal mass matrix consisting of floor masses $[m_b \ m_1 \ m_2 \ m_3]$; $\mathbf{C}$ is the damping matrix based on the Rayleigh’s damping of the fixed-base building with 3% modal damping ratio for its 1st and 2nd modes, where the damping terms from the base isolator and MR damper are set to be zeros in $\mathbf{C}$; $\mathbf{K}$ is the stiffness matrix of the isolated building, where the stiffness contribution from the base isolator and the MR damper is set to be zero; $\mathbf{u} = [u_b \ u_1 \ u_2 \ u_3]^T$ is the displacement vector, where $u_b$, $u_1$, $u_2$, and $u_3$ are the displacements of the base, 1st, 2nd, and 3rd floors, respectively (see Figure 5.1); $\dot{\mathbf{u}}$ and $\ddot{\mathbf{u}}$ are the velocity and acceleration vectors, respectively; $\Lambda = [-1 \ 0 \ 0 \ 0]^T$, $\mathbf{I}$ is the unit vector of which all the rows are 1, and $\ddot{x}_g$ is the input earthquake ground acceleration; $f$ is the sum of the restoring forces from the base isolator and the MR damper, i.e.,
\begin{equation}
  f = f_{BI} + f_{MR}
\end{equation}
where, $f_{BI}$ and $f_{MR}$ are the restoring forces from the base isolator and the MR damper, respectively, which are obtained from equation (2.1) for the base isolator and equation (2.7) for the MR damper. $u_b$ is the displacement of the base floor, but this is also same as the displacements of the base isolator and the MR damper.

The parameters for the hysteretic Bouc-Wen model for the base isolator are provided in Table 5.6, and were determined to be compatible with the design parameters of the bilinear model in Table 5.1.

<table>
<thead>
<tr>
<th>$k_i$ (kN/m)</th>
<th>$k_y$ (kN/m)</th>
<th>$Q_y$ (kN)</th>
<th>$u_y$ (m)</th>
<th>$Q_b$ (kN/m)</th>
<th>$\gamma$</th>
<th>$\beta$</th>
<th>$A$</th>
<th>$m$</th>
</tr>
</thead>
<tbody>
<tr>
<td>7400.0</td>
<td>3333.0</td>
<td>100.0</td>
<td>0.0135</td>
<td>55.0</td>
<td>5.0</td>
<td>5.0</td>
<td>100.0</td>
<td>2.0</td>
</tr>
</tbody>
</table>
The parameters for the MNS MR damper model are provided in Table 5.7 and are
determined to be compatible with the design parameters of the Hershel-Bulkley model in
Table 5.2. It should be noted that the parameters $f_0$ and $C$ are the same as the parameters $a$
and $b$ of equation (2.7), respectively.

<table>
<thead>
<tr>
<th>Input current</th>
<th>$c$ (kN.s/m)</th>
<th>$k$ (kN/m)</th>
<th>$f_0$ (kN)</th>
<th>$C$ (kN.s/m)</th>
<th>$n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I = 0$</td>
<td>90,000</td>
<td>900,000</td>
<td>10</td>
<td>215</td>
<td>1</td>
</tr>
<tr>
<td>$I = I_{\text{max}}$</td>
<td>90,000</td>
<td>900,000</td>
<td>120</td>
<td>925</td>
<td>1</td>
</tr>
</tbody>
</table>

As explained in Chapter 2, the original MNS model has more parameters such as
$\dot{x}_t$ and $m_0$ to more accurately capture the actual MR damper response, but these additional
parameters were set to be zeros in the numerical simulations in this chapter for simplicity.

In order to simulate the electromagnetic dynamics of the MR damper, the variable current
MNS model [94] is used, where the current in the damper coil ($I$) is related to an equivalent
current ($I_{eq}$) through a nonlinear differential equation as

$$
\dot{I}_{eq} = \eta(I_r)(I - I_{eq})
$$

(5.3)

$I_r$ in equation (5.3) is considered to account for the eddy current effect and $\eta(I_r)$ is defined
as

$$
\eta(I_r) = \begin{cases} 
\psi^+ \dot{I}_r + \eta_1; & \text{if } \dot{I}_r \geq 0 \\
\psi^- \dot{I}_r + \eta_1; & \text{if } \dot{I}_r < 0 
\end{cases}
$$

(5.4)

where

$$
\dot{I}_r = \eta_0(I - I_r)
$$

(5.5)
\( \eta_0, \eta_1, \psi^+ \) and \( \psi^- \) in equations (5.4) and (5.5) are constant parameters and are set to be the same as those values provided by Chae et al. [94] for a full-scale MR damper as: \( \eta_0 = 24.96, \eta_1 = 3.57, \psi^+ = 0.31 \) and \( \psi^- = -0.30 \). The use of these values resulted in about 0.9s of rise time for the MR damper in the numerical simulations.

### 5.4.1 Integration Algorithm and Numerical Simulation Modeling

All numerical simulations of this chapter were implemented using the MATLAB/Simulink program [114]. The unconditionally stable explicit CR integration method [115] was used to solve equation (5.1), where the variations in the displacement and velocity vectors of the structure over a time step of \( \Delta t \) are defined as

\[
\begin{align*}
\dot{u}_{i+1} &= \dot{u}_i + \alpha_1 \Delta t \ddot{u}_i \\
u_{i+1} &= u_i + \Delta t \dot{u}_i + \alpha_2 \Delta t^2 \ddot{u}_i
\end{align*}
\]

(5.6) (5.7)

\( u_i, \dot{u}_i \) and \( \ddot{u}_i \) in equations (5.6) and (5.7) are the displacement, velocity, and acceleration vectors of the structure at the \( i \)th time step, respectively. \( \alpha_1 \) and \( \alpha_2 \) are also the matrices of integration parameters defined as [115]

\[
\alpha_1 = \alpha_2 = 4 \left\{ 4M + 2 \Delta t C + \Delta t^2 K \right\}^{-1} M
\]

(5.8)

The Bouc-Wen model was modeled using the built-in Simulink elements, and the differential equations were solved using the internal solver based on the fourth order Runge-Kutta method. The forces from the Bouc-Wen base isolator and the MNS MR damper models are fed into the CR integrator block, and the structural response including the base acceleration \( a_{base} \) is obtained by solving equation (5.1). A low-pass Butterworth filter is then applied to \( a_{base} \) to find \( T_c \) better for implementing the proposed TSA control law of equation (3.1), where the fundamental period of the passive-on isolation system was assumed to be \( T_{iso} \) (i.e., \( T_{iso} = 2.59s \); thus, \( T_{cr} = T_{iso} / \sqrt{2} = 1.83s \). Figure 5.3 shows the
block diagram for the numerical simulations of the adaptively controlled base-isolated structure with an MR controlled by the TSA control law.

**Figure 5.3** Block diagram for numerical simulation of adaptively controlled base-isolated structure with MR damper controlled by the TSA control law

### 5.5 Response Assessment of the Proposed System

The time history response of the designed base-isolated structure is provided in this section to evaluate the performance of the proposed system under various earthquake ground motions. The results from a selected long-period ground motion (1979 Imperial Valley earthquake) are provided with more details and compared with those from a selected short-period ground motion (1989 Loma Prieta earthquake). The results from other short- and long-period ground motions are given in Appendix A.

#### 5.5.1 Response under a Long-Period Ground Motion

Figure 5.4 shows time history responses of the base-isolated building under the 1979 Imperial Valley earthquake which was classified as a long-period earthquake ground motion. As can be observed, the passive-on isolation system performs better than the
passive-off isolation system in terms of reducing the isolator displacement, story drift, velocity, and acceleration demands, implying that the use of high damping is beneficial to reduce structural response under long-period earthquake ground motions. The 3rd story drift in Figure 5.4 represents the relative displacement of the 3rd floor with respect to the 2nd floor displacement. It can be clearly observed that the proposed control law shows a very similar performance to the passive-on isolation system, taking advantage of high damping under the long-period earthquake ground motion. For example, the maximum isolator displacement of the building controlled with the proposed method is 0.257 m, which is almost the same as that of the passive-on isolation system, while much smaller than that of the passive-off isolation system (0.491 m). The results of other floors are similar to the response observed in Figure 5.4.

Figure 5.5 shows the base floor acceleration ($a_{base}$) of the building under the 1979 Imperial Valley earthquake, which was controlled by the proposed TSA control law. As described earlier, a low pass filter needs to be applied to $a_{base}$ in order to minimize the effect of the high-frequency oscillations on the performance of the proposed controller. A 6th order Butterworth filter was applied to $a_{base}$ with a cut-off frequency of 2.5 Hz; this cut-off frequency is about 6.5 times larger than $1/T_{iso}$ (= 0.39 Hz). The low-pass filter introduces a delay of about 0.25 s. If the delay is large, it can diminish the performance of the proposed TSA controller since the controller needs to be implemented in real-time. However, the time delay is much smaller than $T_{iso}$ so that the delay would not significantly impact on the overall performance of the proposed control method. The filtered $a_{base}$ is plotted in Figure 5.5 along with the original unfiltered $a_{base}$. The proposed TSA control law detects $T_c$ with the filtered $a_{base}$, where the first activation of the MR damper occurs
at near 3s, as shown in Figure 5.5. Thereafter, the MR damper remains activated during the strong motion part of the selected long-period earthquake ground motion, making the building work like the passive-on isolation system overall. After the strong motion part, the MR damper is deactivated at around 13.5s, but it is activated again at 16.5s.

Figure 5.6 compares the hysteresis loops of the base isolator and the MR damper for each control case. Larger MR damper force and the increased equivalent damping ratio of the passive-on isolation system effectively reduced the base isolator displacement. Similar to the observation in Figure 5.4, the hysteresis loops from the proposed system are almost the same as those from the passive-on isolation system, making the proposed TSA control law effective under long-period ground motions.
Figure 5.4 Responses of the isolated building under the 1979 Imperial Valley earthquake (long-period): (a) ground acceleration, (b) base isolator displacement, (c) 3rd story drift, (d) 3rd floor absolute velocity, and (e) 3rd floor absolute acceleration
Figure 5.5 Base floor acceleration ($a_{base}$) response of the isolated building with the proposed method under the 1979 Imperial Valley earthquake.

Figure 5.6 Comparison of hysteresis loops of the base isolator and the MR damper for each control case under the 1979 Imperial Valley earthquake.
5.5.2 Response under a Short-Period Ground Motion

Figure 5.7 shows the responses of the base isolated building under the 1989 Loma Prieta earthquake which was classified as a short-period earthquake ground motion. The base isolator displacements observed in the passive-off isolation system are larger than those observed in the passive-on isolation system. The increase of the isolator displacement associated with the use of low damping may not be an issue in this case as long as the isolated building was designed to accommodate the large isolator displacement under long-period excitations. The direction of motion frequently changes under short-period excitations; thus, the amplitudes of the base isolator displacement would be generally small compared to that under long-period excitations. However, the use of high damping will make the system stiffer, transmitting more force to the superstructure under short-period excitations than the case with low damping. For this reason, larger structural responses in the 3rd story drift, 3rd floor absolute velocity, and 3rd floor absolute acceleration are observed in Figure 5.7 when the passive-on isolation system is used, which is quite contrary to the structural responses under the long-period ground motion in Figure 5.4. It is observed that the use of high damping is disadvantageous under short-period earthquake ground motions since it increases structural responses.

While the proposed method worked like a passive-on isolation system under the long-period earthquake ground motion in the previous section, it worked like a passive-off isolation system during the strong motion part of the 1989 Loma Prieta earthquake, as can be observed in Figure 5.7. This can be checked as well in Figure 5.8, where the details how the MR damper was controlled under this selected earthquake ground motion are provided. The MR damper was not activated during the strong motion part because of the high-
frequency contents in $a_{base}$, resulting in the reduced structural responses. However, long-period contents in $a_{base}$ were detected after the strong motion part, and the MR damper was activated at 19.8s and remained activated thereafter.

As can be observed in Figure 5.9, the hysteresis loop of the base isolator of the proposed method is very similar to that of the passive-off isolation system. The hysteresis loop of the MR damper of the proposed method also shows a very similar loop to that of the passive-off isolation system, but some part of the loop has a larger damper force due to the activation of the MR damper after the strong motion part.
Figure 5.7 Responses of the isolated building under the 1989 Loma Prieta earthquake (short-period): (a) ground acceleration, (b) base isolator displacement, (c) 3\textsuperscript{rd} story drift, (d) 3\textsuperscript{rd} floor absolute velocity, and (e) 3\textsuperscript{rd} floor absolute acceleration
Figure 5.8 Base floor acceleration (\(a_{\text{base}}\)) response of the isolated building with the proposed method under the 1989 Loma Prieta Capitola earthquake.

Figure 5.9 Comparison of hysteresis loops of the base isolator and the MR damper for each control case under the 1989 Loma Prieta Capitola earthquake.
5.6 Statistical Analysis and Comparison of Responses

The time history analysis results of the base isolation systems under other earthquake ground motions in Tables 5.4 and 5.5 are provided in Appendix A. The average values for the maximum structural responses of the isolated building under the long-period and short-period earthquake ground motions in Tables 5.4 and 5.5 are compared in Figure 5.10. It can be clearly observed that the use of high damping is very effective to reduce the isolator displacement under the long-period earthquake ground motions. The maximum isolator displacement of the passive-on isolation system is 0.205m, while that of the passive-off isolation system is 0.417m; there is about 51% reduction in the isolator displacement, which can significantly reduce the pounding risk of the base-isolated building associated with large isolator displacements. High damping is also effective in reducing the floor velocity under long-period earthquake ground motions. The maximum 3rd floor velocity was reduced from 1.343 m/s to 1.038 m/s when the damper was fully activated compared to the case of the passive-off isolation system. The floor velocity is related to the kinetic energy of the unfixed objects on the floor, which can cause a large impact force when a collision occurs among objects. Thus, high damping can reduce this kind of risk in buildings such as hospitals that have various unfixed medical utilities and objects. The story drift and acceleration responses were slightly increased with the passive-off isolation system, but no significant difference is observed between the passive-on and passive-off isolation systems under the long-period ground motions. Overall, it is observed that the results from the proposed isolation system are very similar to those from the passive-on isolation system under the long-period ground motions.
Under the short-period earthquake ground motions, high damping is also effective in reducing the isolator displacement. However, it should be noted that the isolator displacements under short-period ground motions are much smaller than those under long-period ground motions. Therefore, in general, the increased isolator displacement using low damping (i.e., the passive-off isolation system) would not be an issue as long as the isolator is designed to accommodate the large displacement under strong long-period ground motions. It is observed that low damping is effective to reduce the story drift, floor velocity, and floor acceleration under the short-period ground motions, which is consistent with the observation from the transmissibility theory in Figure 3.1. The 3rd story drift and 3rd floor velocity were reduced by about 30% and 22%, respectively, by using the passive-off system. The amplitudes of the floor velocity under the short-period ground motions are much smaller than those under the long-period ground motions. In terms of the acceleration response, the passive-off isolation system reduced the maximum acceleration by about 33% compared with the passive-on isolation system (i.e., from 0.37g to 0.25g). Overall, it is observed that the use of low damping is advantageous under the short-period ground motions, and the proposed control law made the isolation system similar to the passive-off isolation system under the short-period ground motions.
Figure 5.10 Average of maximum structural responses of base-isolated three-story building. (a) under long-period ground motions, (b) under short-period ground motions

Acceleration response is closely related to the damage of nonstructural components, particularly for fixed nonstructural components. The maximum value of the acceleration response is also important, but the floor spectral acceleration is more useful to understand the impact of damping on the response of nonstructural components. Figure 5.11 compares the average spectral accelerations of the three different control methods at the 3rd floor under the long-period and short-period ground motions. Basically, high damping is effective to reduce the spectral acceleration around the fundamental periods of the isolated buildings. On the other hand, the use of low damping is observed to be beneficial to reduce the spectral acceleration at short periods less than 1s under both short- and long-period earthquake ground motions. It should be noted that the response of nonstructural components is sensitive to the spectral acceleration at short-periods since the natural periods of nonstructural components are generally short. It can be observed that the short-period spectral accelerations under the short-period ground motions are larger than
those under the long-period ground motions, implying that nonstructural components will be subjected to a more critical condition under strong short-period ground motions. In particular, the short-period spectral accelerations were more amplified under the short-period ground motions with the use of high damping. Therefore, low damping needs to be used for a base isolated structure under short-period earthquake ground motions to effectively reduce nonstructural component damage.

Figure 5.11 Comparison of average spectral acceleration at the 3rd floor of building: (a) under long-period ground motions, (b) under short-period ground motions

From Figures 5.10 and 5.11, it can be clearly observed that the base isolation system with the proposed TSA control law performs very similar to the passive-on isolation system under long-period earthquake ground motions, while working like the passive-off isolation system under short-period earthquake ground motions. The proposed system can take advantage of both passive-on and passive-off systems in mitigating the seismic hazard of structures, which can assure better the high-performance resiliency under various earthquake ground motions with a wide range of frequency contents.
5.7 Summary

In this chapter, a base-isolated three-story building with MR dampers was designed by implementing the simplified design procedure developed in Chapter 4. Basically, the isolated building was designed to be a passive isolation system with its maximum damping to minimize the isolator displacement and the structural response under strong long-period ground motions. Then, the MR damper is deactivated whenever there is a short-period content satisfying the deactivation condition to minimize the damage of structural and nonstructural components. The performance of the proposed TSA control law was validated by conducting nonlinear time history analyses (NTHA) with selected long- and short-period earthquake ground motions. A base isolator based on the Bouc-Wen hysteresis model and the MNS MR damper model was used in the numerical simulations. Parameters for these models were determined based on the simplified design procedure, where the base-isolated building can have the maximum equivalent damping ratio of $\zeta_{eq}=40\%$ and the minimum equivalent damping ratio of $\zeta_{eq}=10\%$ by controlling the input current to the MR damper. The numerical simulations clearly showed that high damping is effective to reduce the isolator displacement and structural responses under long-period earthquake ground motions, while the protection of nonstructural components can be effectively achieved by using low damping, especially under strong short-period earthquake ground motions. This means that it is difficult to satisfy high-level seismic performance objectives for a given building by simply using passive isolation systems. Unlike these passive isolation systems, it was demonstrated that the proposed isolation system can resolve this issue; the proposed TSA control law makes the building work like a passive-on or passive-off isolation system as necessary to achieve high-performance level under both long-and
short-period earthquake ground motions, which can significantly improve the resiliency and sustainability of buildings.
CHAPTER 6
REAL-TIME HYBRID SIMULATION OF THE ADAPTIVE TSA BASE ISOLATION SYSTEM

6.1 General

In order to accurately understand the performance of seismic protection systems under earthquake loading, experimental studies should be conducted. Effective force testing (EFT) and shake table testing are the two dynamic testing methods that can consider the rate-dependency of test structures, but they require the entire structural system to be constructed in the laboratory. In comparison to the EFT and shake table testing, real-time hybrid simulations (RTHSs) are much more efficient. RTHS does not need the construction of the whole structure in the lab. It just requires the structural component of interest (experimental substructure), and the remaining parts of the structure are modeled numerically (analytical substructure), which makes this type of testing economic. Additionally, various earthquake ground motions can be applied to the structure of interest through RTHS tests, as long as the test structure does not have a failure, easily enabling the statistical analysis of the results. Due to these advantages, RTHS has been implemented by numerous researchers in recent years [2, 95, 116-123].

Due to the complex nonlinear behavior of the combined system of base isolators and MR dampers integrated with the proposed TSA controller, experimental studies should be conducted to more accurately assess the performance of the proposed isolation system under earthquake ground motions. In this chapter, RTHSs are performed on a small-scale base-isolated building with a rubber isolator and a small-capacity MR damper to experimentally evaluate the performance of the TSA controller. As explained in Chapter
3, the TSA controller makes a decision based on the acceleration response at the base of
the structure to increase the damping under long-period earthquakes, while keeping the
damping minimum under short-period excitations. In RTHSs of this chapter, the structure
above the isolation system is modeled numerically (numerical substructure) and designed
to accommodate the restrictions existing in the experimental substructure (base-isolation
system). The simplified design procedure introduced in Chapter 4 is used for the design of
the small-scale three-story base-isolated building with MR dampers. The base-isolation
system consists of a small-scale rubber bearing combined with linear bearings and an MR
damper.

Characterization tests are conducted on both the MR damper and the rubber bearing
combined with the linear bearings. These tests are for accurately identifying the parameters
of the numerical models for the MR damper, rubber bearing combined with the linear
bearings. The RTHS results are compared with the numerical simulation results. Statistical
results under various earthquake ground motions are also provided to better assess the
performance of the proposed system.

6.2 Structural System

The structural system considered in this chapter is a three-story base-isolated
building which is schematically shown in Figure 6.1. The base isolation system consists of
base isolators, linear bearings, and MR dampers. The combination of the rubber bearing
and the linear bearings provides similar performance to a base isolation system with
crossed linear bearing (CLB) [124]. The structural system above the isolation layer is
assumed to behave like a typical shear building.
6.3 Experimental test setup

As shown in Figure 6.2, the experimental substructure consists of one small-scale rubber bearing base isolator, three linear bearings, one MR damper (connected to a current driver), the servo-hydraulic actuator with the load cell, the reaction frame, and the tie-side steel box. The rubber bearing is manufactured by Dynamic Isolation Systems Incorporation and consisted of a 6-inch diameter rubber with a height of 6.5 inches attached to two 9 by 9 inches square base plates. Each base plate has a thickness of 0.75 inches and provides with four bolt holes of 0.75-inch diameter at the corners for connection purpose. The rubber bearing is connected to the three unidirectional linear bearings with a length of 17.875 inches at the top and is fixed at the bottom to the base of the tie-side steel box. This setup
provides a unidirectional motion of the rubber bearing along the longitudinal axis of the actuator. One end of a small force capacity MR damper is directly attached to the rubber bearing, and the other end is attached to the reaction frame. The MR damper is manufactured by the Lord Corporation (RD-8041-1) with a maximum stroke of 74mm. The damping force of the MR damper is dependent on the damper’s piston velocity. At the velocity of 50mm/s with a current input of 1 A, the small-scale MR damper can provide at least 2.447kN of damping force. In order to properly control the current to the MR damper, an analog pulse-width-modulation (PWM) servo drive (AZB20A8) is used. The PWM servo drive is mounted on an AZ drive mounting card (MC1XAZ01), both manufactured by Advanced Motion Control. The PWM servo drive is designed to drive a direct-current (DC) motor at a high switching frequency and can supply continuous current up to 12 A. This current drive system can minimize the inductance in the electromagnetic coil of the MR damper and quickly switch the command current sent by the controller. The maximum force capacity and stroke limit of the actuator (MTS Model 244.12) are 25kN and 152.4mm, respectively. The attached load cell to the actuator is used to measure the restoring force of the base isolation system (i.e., the total force from the MR damper, rubber bearing, and linear bearings). An accelerometer from PCB Piezotronics (393B04 model) is attached to the top of the rubber bearing, as shown in Figure 6.2, to measure acceleration at the base of the isolated structure.
The reaction frame has pre-existing bolt holes to enable various test setups to be accommodated within the frame, where the steel box was secured using these bolt holes. Since the bolt holes were not placed to have the pistons of the actuator and MR damper on
the zero position, the assembled setup for the base isolation system of Figure 6.2 has a maximum stroke limit of 50mm although each component of the setup (i.e., MR damper, rubber bearing and linear bearings) can provide a larger stroke limit individually as explained before. The limitations in the force and velocity capacity of the servo-hydraulic actuator system, along with the stroke limit of the setup, were considered in the design of the superstructure above the isolation system. Thus, a small-scale three-story building was designed to accommodate these restrictions in the experimental tests of this study.

6.4 Characterization Tests

Characterization tests were conducted for the rubber bearing combined with the linear bearings and the MR damper in Figure 6.2. In order to characterize the rubber bearing combined with the linear bearings, the MR damper was removed first from the entire experimental setup. The rubber bearing combined with the linear bearings was then subjected to a sinusoidal wave. After the characterization test, the tie-side steel box including rubber bearing and linear bearings was removed and the MR damper was directly connected to the actuator for performing characterization tests under various constant current inputs. In all these tests, a quasi-static sinusoidal wave was slowly imposed with an excitation frequency of 0.1Hz as shown in Figure 6.3. The amplitude of the sinusoidal wave gradually increases with a slope of 1.13mm/s to reach the maximum stroke of the experimental test setup (i.e. ±25mm).
Figure 6.3 Characterization tests: (a) input sine wave, (b) force-displacement relationship of rubber bearing combined with linear bearings, (c) force-displacement relationship of MR damper under maximum input current (i.e. 1.5 A)

It can be clearly observed from Figure 6.3 (b) and (c) that both the rubber bearing combined with the linear bearings and the MR damper are highly nonlinear. Figure 6.4 shows the force-displacement response of the MR damper under sinusoidal displacement input with various current input values.
Figure 6.4 Force-displacement response of MR damper under sinusoidal displacement input for various constant current inputs (frequency=0.1Hz, amplitude=25mm)

Figure 6.5 provides the response of the whole base isolation system (i.e., total restoring force from the rubber bearing, linear bearings, and MR damper) under the same displacement input of Figure 6.3(a). The current inputs to the MR damper are minimum and maximum, i.e. 0 A and 1.5 A, respectively.
6.4.1 Identification of Parameters for Simplified Design Procedure

From the characterization tests, the coefficients for the Herschel-Bulkley MR damper model and the bilinear rubber bearing model were identified, which are given in Table 6.1. These coefficients will be used for the design of a small-scale three-story base-isolated building using the simplified design procedure.

<table>
<thead>
<tr>
<th>Herschel-Bulkley MR damper model</th>
<th>Input current</th>
<th>$f_0$ (kN)</th>
<th>$C$ (kN⋅s/m)</th>
<th>$n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I = I_{\text{max}}$ (passive-on)</td>
<td>1.10</td>
<td>11.0</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>$I = 0$ (passive-off)</td>
<td>0.04</td>
<td>1.0</td>
<td>1.0</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Rubber bearing bilinear model</th>
<th>$Q_y$ (kN)</th>
<th>$x_y$ (m)</th>
<th>$k_y$ (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.15</td>
<td>0.00001875</td>
<td>118.0</td>
</tr>
</tbody>
</table>
In Table 6.1, $f_0$ is the damper force at the maximum displacement of $x_0$; also $C$ and $n$ are the parameters characterizing the Herschel-Bulkley MR damper model. These coefficients are for the maximum and minimum currents for the MR damper. $Q_y$, $x_y$, and $k_y$ are the initial yield force, initial yield displacement, and yield stiffness of the rubber bearing, respectively. The frictional force of the linear bearings is measured to be 0.075kN.

### 6.4.2 Identification of Parameter for the MNS Model and Bouc-Wen Model

For modeling the response of the small-scale MR damper, the Maxwell nonlinear slider (MNS) model [93, 94] is used in this chapter. The identified parameters for the MNS model of the small-scale MR damper is provided in Table 6.2 for various constant current inputs based on the characterization tests.

**Table 6.2 Parameters for the MNS MR damper model**

<table>
<thead>
<tr>
<th>Input current</th>
<th>$c$ (kN·s/m)</th>
<th>$k$ (kN/m)</th>
<th>$f_0$ (kN)</th>
<th>$C$ (kN·s/m)</th>
<th>$n$</th>
<th>$\dot{x}_T^+$ (m/s)</th>
<th>$f_0$ (kN)</th>
<th>$C$ (kN·s/m)</th>
<th>$n$</th>
<th>$\dot{x}_T^-$ (m/s)</th>
<th>$m_0$ (kN·s²/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I=0.0A</td>
<td>7,000</td>
<td>1,945</td>
<td>0.04</td>
<td>0.95</td>
<td>0.9</td>
<td>6.0e-6</td>
<td>-0.02</td>
<td>-1.0</td>
<td>1.0</td>
<td>-9.0e-6</td>
<td>-0.15</td>
</tr>
<tr>
<td>I=0.3A</td>
<td>7,100</td>
<td>1,950</td>
<td>0.3</td>
<td>4.5</td>
<td>0.5</td>
<td>5.6e-3</td>
<td>-0.1</td>
<td>-2.6</td>
<td>0.3</td>
<td>-4.9e-3</td>
<td>-0.01</td>
</tr>
<tr>
<td>I=0.6A</td>
<td>7,150</td>
<td>1,960</td>
<td>0.8</td>
<td>5.0</td>
<td>0.6</td>
<td>7.6e-3</td>
<td>-0.5</td>
<td>-2.6</td>
<td>0.3</td>
<td>-5.8e-3</td>
<td>-0.09</td>
</tr>
<tr>
<td>I=0.9A</td>
<td>7,200</td>
<td>1,950</td>
<td>0.9</td>
<td>5.5</td>
<td>0.6</td>
<td>7.4e-3</td>
<td>-0.6</td>
<td>-2.6</td>
<td>0.3</td>
<td>-6.5e-3</td>
<td>-0.08</td>
</tr>
<tr>
<td>I=1.2A</td>
<td>7,250</td>
<td>1,980</td>
<td>1.0</td>
<td>8.0</td>
<td>0.7</td>
<td>7.5e-3</td>
<td>-1.0</td>
<td>-3.5</td>
<td>0.5</td>
<td>-7.1e-3</td>
<td>-0.05</td>
</tr>
<tr>
<td>I=1.5A</td>
<td>7,300</td>
<td>2,000</td>
<td>1.1</td>
<td>11</td>
<td>0.8</td>
<td>5.5e-3</td>
<td>-1.1</td>
<td>-5.0</td>
<td>0.6</td>
<td>-5.4e-3</td>
<td>-0.05</td>
</tr>
</tbody>
</table>

Detailed information about the MNS model parameters is provided in Chapter 2. In order to model the electromagnetic dynamics of the MR damper, the variable current MNS model [94] is used. Equations (5.3) to (5.5) in Chapter 5 represent the nonlinear differential equations to be solved for finding the equivalent current to the MR damper. The constant coefficients in equations (5.4) and (5.5) are determined from the current step up (from 0 A
to 1.5 A) and step down (from 1.5 A to 0 A) characterization tests as $\eta_0=60$, $\eta_1=150$, $\psi^+=7$ and $\psi^-=7$.

The response of the rubber bearing is modeled using the Bouc-Wen model [81]. The linear bearings also provide a frictional force which is modeled based on the Coulomb’s friction model. Figure 6.6 shows the friction model for the linear bearings.

![Figure 6.6 Linear-bearing Coulomb’s friction model](image)

The frictional force of the linear bearings is determined by the following equation as

$$f_{LB} = Q_0 \, sgn(\dot{x}_b) \tag{6.1}$$

where, $\dot{x}_b$ is the velocity at the base of the structure, $sgn()$ is the signum function and $Q_0$ is the friction force. The parameters for the Bouc-Wen model and the Coulomb’s frictional element are provided in Table 6.3.
Table 6.3 Parameters of the Bouc-Wen model and the Coulomb’s frictional element

<table>
<thead>
<tr>
<th></th>
<th>Bouc-Wen Hysteresis model</th>
<th>Coulomb’s element</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k_i$ (kN/m)</td>
<td>$k_y$ (kN/m)</td>
<td>$Q_y$ (kN)</td>
</tr>
<tr>
<td>8000.0</td>
<td>118.0</td>
<td>0.15</td>
</tr>
</tbody>
</table>

Detailed information for the parameters of the Bouc-Wen hysteresis model in Table 6.3 is provided in Chapter 2. The restoring force from the base isolation system is a combination of restoring forces from the MR damper, rubber bearing and linear bearings, which is defined as

$$F^e = f_{MR} + f_{RB} + f_{LB}$$  \hspace{1cm} (6.2)

$F^e$ is measured by the load cell attached to the actuator (shown in Figure 6.2) during RTHS. Figure 6.7 shows the comparison of the experimental response of the base isolation system with the numerical simulation, where the sine wave used in the characterization tests was applied to the combined base isolation system. It can be clearly observed that the force and displacement time histories along with the force-displacement relationship from the numerical simulation match well with the experimental results under the sinusoidal wave input.
Figure 6.7 Comparison of numerical simulation results with experimental test results of the combined base isolation system under the sinusoidal input displacement (input current to the MR damper = 1.5 A): (a) input sinusoidal wave, (b) force time history, (c) force-displacement relationship

6.5 Selection of Earthquake Ground Motions

Tables 6.4 and 6.5 provide the selected earthquake ground motions for the long- and short-period motions, respectively. Generally, the effective period of typical low-rise base-isolated buildings is between 2.0s to 4.0s. It was assumed that the prototype base isolation building has an effective period of 3.0s in this chapter, while the effective period of the corresponding fixed based structure is 1.0s. Similar to the ground motion selection procedure in Chapter 5, an earthquake ground motion is classified as a short-period motion if the ratio of the spectral acceleration at the fundamental period of isolated structure to that of the structure without base isolation is less than 0.4; otherwise, it is classified as a long-period motion. Figure 6.8 shows the response spectra of the original earthquake ground motions in Tables 6.4 and 6.5 without any scale factor applied.
### Table 6.4 Long-period earthquake ground motions

<table>
<thead>
<tr>
<th>ID</th>
<th>Earthquake</th>
<th>Year</th>
<th>Station</th>
<th>Magnitude</th>
<th>Component</th>
<th>Scale Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Imperial Valley</td>
<td>1979</td>
<td>El Centro Array No. 6</td>
<td>6.5</td>
<td>230</td>
<td>0.27</td>
</tr>
<tr>
<td>2</td>
<td>Northridge</td>
<td>1994</td>
<td>Sylmar Converter</td>
<td>6.7</td>
<td>052</td>
<td>0.29</td>
</tr>
<tr>
<td>3</td>
<td>Chi-Chi, Taiwan</td>
<td>1999</td>
<td>CHY101</td>
<td>6.3</td>
<td>E</td>
<td>0.41</td>
</tr>
<tr>
<td>4</td>
<td>Northridge</td>
<td>1994</td>
<td>Jensen</td>
<td>6.7</td>
<td>022</td>
<td>0.40</td>
</tr>
<tr>
<td>5</td>
<td>Manjil, Iran</td>
<td>1990</td>
<td>Abbar</td>
<td>7.4</td>
<td>Transverse</td>
<td>0.69</td>
</tr>
</tbody>
</table>

### Table 6.5 Short-period earthquake ground motions

<table>
<thead>
<tr>
<th>ID</th>
<th>Earthquake</th>
<th>Year</th>
<th>Station</th>
<th>Magnitude</th>
<th>Component</th>
<th>Scale Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Loma Prieta</td>
<td>1989</td>
<td>Capitola CDMG 47125</td>
<td>6.93</td>
<td>000</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>San Fernando</td>
<td>1971</td>
<td>LA Hollywood Stor Lot</td>
<td>6.61</td>
<td>180</td>
<td>1.0</td>
</tr>
<tr>
<td>3</td>
<td>Cape Mendocino</td>
<td>1992</td>
<td>Rio Del Overpass</td>
<td>7.01</td>
<td>360</td>
<td>1.0</td>
</tr>
<tr>
<td>4</td>
<td>Friuli, Italy</td>
<td>1976</td>
<td>Tolmezzo</td>
<td>6.50</td>
<td>000</td>
<td>1.0</td>
</tr>
<tr>
<td>5</td>
<td>Northridge</td>
<td>1994</td>
<td>Canyon Country</td>
<td>6.69</td>
<td>000</td>
<td>1.0</td>
</tr>
</tbody>
</table>

![Figure 6.8 Response spectra of the original earthquake ground motions](image)

As explained earlier, the isolation system of this chapter has a relatively small capacity in the maximum displacement (less than 25mm). Therefore, a base-isolated
structure with the isolation system of Figure 6.2 would not be able to accommodate the large isolation displacement expected at the natural period of 3.0s under the original long-period ground motions of Table 6.4. Moreover, the large spectral accelerations of long-period ground motions at longer periods further increase the displacement demand of the base isolation system, which needs to be appropriately addressed in the experimental setup with a limited displacement capacity.

In order to resolve these issues, the time axis of all earthquake ground motions was scaled down with a scale factor of 1/3 and the effective period of the isolated building to be used in the experimental study of this chapter is assumed to be 1.0s. Furthermore, the accelerations of all long-period earthquake ground motions were additionally scaled down with scale factors less than 1.0, as provided in Table 6.5. These will enable the experimental investigation of the performance of the proposed base isolation system under both long- and short-period ground motions within the given displacement capacity of the isolation system. The accelerations of the short-period earthquake ground motions were not scaled as shown in Table 6.5 since their spectral accelerations around the fundamental period of the base-isolated structure are generally small, resulting in a small displacement demand in the isolator.
Figure 6.9 shows the response spectra of scaled earthquake ground motions of Tables 6.4 and 6.5 with a 5% damping ratio. It can be verified that long-period earthquake ground motions have much larger averaged spectral accelerations around the effective period (i.e., 1.0s) of the isolation systems than short-period earthquake ground motions.

### 6.6 Design of a Small-Scale Three-Story Base Isolated Building

The procedure for designing a small-scale three-story base-isolated building with the MR damper is described in this section. The simplified design procedure introduced in Chapter 4 is implemented, and the maximum isolator displacement $x_0$ and the maximum base shear $V_{max}$ of the isolation system are determined to satisfy the given restrictions of the test setup (e.g., the stroke limit of the test setup and the maximum force capacities of the actuator, rubber bearing and MR damper). The effective stiffness of the base isolation system is determined from the characterization tests. Based on the command signal from the TSA controller, the current input to the MR damper is switched between the maximum
and minimum values. Given the effective stiffness of the passive-on base isolation system \( k_{\text{eff,BIS}} \) and the total mass of the isolated building as \( m_t \), the effective period of the isolation system \( T_{\text{eff}} \) is determined based on the formulation provided by the simplified design procedure as:

\[
T_{\text{eff}} = 2\pi \sqrt{\frac{m_t}{k_{\text{eff,BIS}}}}
\]  

(6.3)

From the characterization tests, the effective stiffness of the passive-on isolation system was identified to be \( k_{\text{eff,BIS}}=160 \) kN/m. Considering the limitation of the displacement capacity of the base isolator test setup, the effective period of the isolated building of this study is assumed to be 1.0s. This results in the total mass \( m_t \) being 4 kN.s\(^2\)/m. It was assumed that each floor has the same mass; thus, \( m_b = m_1 = m_2 = m_3 = 1 \) kN.s\(^2\)/m. The story stiffness was determined to have the fundamental natural period of the passive-off system to be close to 1.0s, resulting in \( k_1 = 3,037 \) kN/m, \( k_2 =2,336 \) kN/m and \( k_3 =1,635 \) kN/m. Table 6.6 shows the natural periods of the three different structural systems using the structural properties above.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Period ( T_n ) (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No isolation (fixed-base)</td>
</tr>
<tr>
<td>1</td>
<td>0.28</td>
</tr>
<tr>
<td>2</td>
<td>0.11</td>
</tr>
<tr>
<td>3</td>
<td>0.07</td>
</tr>
</tbody>
</table>

The effect of isolation in increasing the natural period of the structure can be clearly observed based on the values provided in Table 6.6 (i.e. from 0.28s for the fixed-base
structure to 0.92s for the isolated structure in passive-on mode). The natural period of the small-scale structure in passive-on mode is less than that of the passive-off mode, which is due to the increased stiffness of the base isolation system with passive-on mode.

The maximum displacement $x_0$ of the isolation system was checked for each long-period earthquake ground motion in Table 6.4, and it was found that the maximum $x_0$ is $x_{0,\text{max}} = 23.7\text{mm}$ under the 1979 Imperial Valley earthquake ground motion, which is less than the maximum displacement capacity of the isolation system (i.e., 25mm). The maximum base shear $V_{\text{max}}$ was found to be 3.203 kN, which is less than the maximum actuator capacity of 25kN. The equivalent damping ratios of the passive-on system with the averaged response spectra of the long-period and short-period ground motions are 57.31% and 55.78%, respectively. The equivalent damping ratios of the passive-off system with the averaged response spectra of the long-period and short-period ground motions are 7.93% and 11.78%, respectively.

### 6.7 Real-Time Hybrid Simulations for the Proposed Base Isolation System

RTHS consists of experimental and numerical substructures, the integration algorithm, the servo-hydraulic actuator system, and the associated controllers. A stable and accurate integration algorithm is necessary for calculating the displacement response during RTHS. The calculated displacement response of the integration algorithm is called the target displacement. The servo-hydraulic actuator system should be accurately and reliably controlled to correctly impose the target displacement on the experimental substructure. In order to secure the stability of the system in real-time, communication among the numerical substructure, integration algorithm and servo-hydraulic actuator system must be implemented with minimum delay [125]. In order to minimize time delays
and amplitude differences between the target and measured displacements of the experimental substructure, actuator delay compensators need to be implemented, where the output displacement of the compensator, i.e., the compensated displacement, is directly applied to the experimental substructure.

### 6.7.1 Governing Equations of Motion

In the small-scale three-story base-isolated structure of this chapter, the base isolation system is considered as the experimental substructure and the structure above the isolation system is modeled numerically and considered as the numerical substructure. In this section, the governing differential equations of the system and the procedure for solving this equation in real-time is explained. The discretized equations of motion of the base-isolated three-story building with MR damper shown in Figure 6.1 at the $i+1$th time step are given as

$$M\ddot{x}_{i+1} + C\dot{x}_{i+1} + Kx_{i+1} = \lambda F^e_{i+1} - M\Pi\ddot{g}_{i+1}$$

(6.4)

$x_{i+1}$ is the displacement vector; $\dot{x}_{i+1}$ and $\ddot{x}_{i+1}$ are the velocity and acceleration vectors of the structure, respectively; $K$ and $C$ are the stiffness and damping matrices of the structure, respectively; where the stiffness and damping contribution of the base isolation system (i.e., experimental substructure) is excluded and set to be zero; $M$ is the diagonal mass matrix of structure; $\lambda = [-1 \ 0 \ 0 \ 0]^T$, $\Pi$ is a unit vector of all rows equals to 1, $\ddot{g}_{i+1}$ is the earthquake ground acceleration; $F^e_{i+1}$ is the experimental substructure restoring force measured by the load cell which represents the summation of restoring forces from the MR damper, rubber bearing and linear bearings. In order to solve equation (6.4) in real-time, the CR integration algorithm [115] was used. MATLAB Simulink module is used to model the numerical substructure, TSA controller and integration algorithm [114]. The block
diagram for the RTHS of the semi-actively controlled base-isolated structure with MR damper is shown in Figure 6.10.

It should be noted that since the restoring force from the experimental substructure is added to the right side of Equation (6.4), its contribution should be excluded from the numerical part to prevent the double consideration. This is the reason why the stiffness and damping contribution of the experimental substructure is set to be zero into the stiffness matrix $\mathbf{K}$ and damping matrix $\mathbf{C}$ in the numerical substructure.

As explained previously, the acceleration at the base of the structure is measured by the accelerometer attached to the experimental substructure. Since the TSA controller utilizes the absolute acceleration, the ground acceleration is added to the measured acceleration by the accelerometer. This procedure is clearly shown in the block diagram of Figure 6.10. Then, a Butterworth low-pass filter is applied to the combined acceleration data to remove the noise and high-frequency oscillations. The TSA controller makes a
decision based on the filtered acceleration data and sends the command to the current driver. The input current to the MR damper will be received from the current driver.

Due to the inherent nonlinearities of the servo-hydraulic actuator system and the nonlinear response of the experimental substructure, time delays and amplitude changes in the actuator is inevitable during real-time testing [126-128]. In this dissertation, the Adaptive Time Series (ATS) compensator developed by Chae et al. [129] is used to compensate for the actuator control error. In addition, instability or very poor control of the system can occur in RTHS if the stiffness of the whole structure (i.e., numerical and experimental substructures) was not considered in the integration algorithm parameters. It was shown in previous research conducted by Chen and Ricles, and Chae et al. [2, 115] that the solution will remain stable as long as the total damping and stiffness in the integration parameters are larger than those developed in the system during RTHS. Thus, the initial stiffness of experimental substructure (10,000 kN/m calculated based on the characterization tests) was added to the stiffness matrix of the whole structure at the base level in the parameter calculation of CR integration algorithm. In terms of damping, it was observed that the stability of the system was not very sensitive to the values of equivalent damping.

6.7.2 Real-Time Control System Architecture

The real-time control system of the servo-hydraulic actuators in the Structural Laboratory at Old Dominion University is used for implementing the TSA control algorithm proposed in this dissertation. The architecture of this system is shown in Figure 6.11. In order to run the system in real-time, the model and the TSA control algorithm simulated in the simulation PC of Figure 6.10 will be compiled into the xPC and the
command signals to the servo-hydraulic actuators will be sent through a Shared Common Random Access Memory Network (SCRAMNet). Since the servo-hydraulic actuators are controllable directly by the MTS FlexTest 40 controller, SCRAMNet, which works as a real-time data communication bus, makes it possible to impose the target displacement to a test structure via MTS FlexTest using the actuators. Moreover, receiving the feedback data (i.e. displacement and force) from the actuator through the MTS FlexTest controller and sharing them to the xPC becomes possible through the SCRAMNet. The response of the experimental substructure can be collected via the data acquisition system through the Input/Output board from Speedgoat (IO-102), as well. For this purpose (measuring the response of test structure), different voltage-based sensors (e.g., accelerometer, displacement transducer, and load cell) attached to the test structure can be plugged into the data acquisition system through its 32 analog input channels with 16-bit data module. The data acquisition board also consists of 8 digital inputs, 8 digital outputs, and 4 analog outputs. Measured data from the test structure or command signals to the board will also be directly transferred to the xPC. The entire real-time control system mentioned herein works with a data sampling rate of 1024Hz.
6.7.3 Actuator Delay Compensation

The MTS FlexTest 40 used for the actuator control has a proportional-integral-derivative (PID) controller. In addition to this PID controller, the ATS compensator is implemented in the real-time hybrid simulations of this study to minimize the effect of actuator delay and possible amplitude changes as described previously. Although the base-isolation substructure of this study is much more flexible than the rigid system tested by Chae et al. [130] in a real-time manner using the same servo-hydraulic actuator system, the high-frequency oscillation at the oil-column resonance frequency of the system is still observed at the beginning of some tests in the passive-on mode. In order to resolve this issue and to have a good control of the system during the test, the P- and I-gains were set to be 12.0 and 1.0, respectively, and the D-gain set to be zero in the PID controller. In the
ATS compensator, the compensated command displacement to the actuator ($x^c$) is obtained as:

$$x^c = a_0 x^t + a_1 \dot{x}^t + a_2 \ddot{x}^t$$  \hspace{1cm} (6.5)

where $x^t$, $\dot{x}^t$, and $\ddot{x}^t$ are the target displacement, velocity, and acceleration, respectively. $a_0$, $a_1$, and $a_2$ are the system coefficients which are updated during RTHS in order to minimize the error between the measured and target displacements. The coefficients for the ATS compensator are provided in Table 6.7.

<table>
<thead>
<tr>
<th>Coefficient</th>
<th>Initial Values</th>
<th>Range (minimum, maximum)</th>
<th>Maximum rate of change</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a_0$</td>
<td>1.0</td>
<td>(0.50, 2.0)</td>
<td>2/s</td>
</tr>
<tr>
<td>$a_1$</td>
<td>0.022</td>
<td>(0.0, 0.10)</td>
<td>0.05 s/s</td>
</tr>
<tr>
<td>$a_2$</td>
<td>0.0008</td>
<td>(0.0, 0.01)</td>
<td>0.001 s^2/s</td>
</tr>
</tbody>
</table>

### 6.8 RTHS Results under Long- and Short-Period Earthquake Ground Motions

The time history responses of the base-isolated structure under the 1979 Imperial Valley earthquake (long-period) and the 1976 Friuli, Italy earthquake (short-period) are shown in Figures 6.12 and 6.13, respectively, for the proposed (i.e., base isolation system controlled with the TSA controller), passive-on and passive-off base isolation systems. It is desirable to have high and low damping under long- and short-period earthquake ground motions, respectively. High damping is effective in reducing structural responses, especially the isolator displacement under long-period motions; however, it will increase the story drift and acceleration demand under short-period ground motions [9]. It can be clearly observed from Figures 6.12 and 6.13 that the proposed control system works similar
to the passive-on isolation system taking the advantage of using high damping under long-period ground motions, while working similar to the passive-off system with low damping under short-period ground motions.
Figure 6.12 Time history responses under the 1979 Imperial Valley earthquake (long-period) for the isolated building: (a) ground acceleration, (b) base isolator displacement, (c) 3rd story drift, (d) 3rd floor absolute velocity, and (e) 3rd floor absolute acceleration
Figure 6.13 Time history responses under the 1976 Friuli, Italy earthquake (short-period) for the isolated building: (a) ground acceleration, (b) base isolator displacement, (c) 3\textsuperscript{rd} story drift, (d) 3\textsuperscript{rd} floor absolute velocity, and (e) 3\textsuperscript{rd} floor absolute acceleration.
The base floor acceleration \(a_{\text{base}}\) of the structure under the 1979 Imperial Valley earthquake (long-period) and the 1976 Friuli, Italy earthquake (short-period) is shown in Figures 6.14 and 6.15, respectively. It should be noted that \(a_{\text{base}}\) is the absolute acceleration at the base of the building. Based on the fundamental period of the passive-on isolation system (i.e., \(T_{\text{iso}}=0.92\text{s}\)), \(T_{\text{cr}}\) is determined to be \(T_{\text{cr}} = T_{\text{iso}}/\sqrt{2} = 0.65\text{s}\). \(T_{\text{GT}}\) is considered to be the same as \(T_{\text{iso}}\), and \(\alpha_1\) and \(\alpha_2\) are set to be 0.78 and 0.2, respectively.

As previously described, in order to minimize the effect of noise and high-frequency oscillations, a low-pass filter is applied to \(a_{\text{base}}\) to improve the performance of the TSA controller. A 6th order Butterworth filter with a cut-off frequency of 10Hz was applied to \(a_{\text{base}}\), which is about 10 times larger than \(1/T_{\text{iso}}\) (=1.087Hz). A delay of about 0.062s will be introduced to the acceleration using the low-pass filter, which is much smaller than \(T_{\text{iso}}\) and will not have a significant effect on the overall performance of the controller. The unfiltered and filtered \(a_{\text{base}}\) data are plotted in both Figures 6.14 and 6.15. The proposed TSA controller makes a decision on sending the activation signal to the MR damper based on the detection of \(T_c\) from the filtered \(a_{\text{base}}\), where the first activation occurs at near 1.0s under the 1979 Imperial Valley earthquake (Figure 6.14) and the damper remains on throughout the rest of motion. Under the 1976 Friuli earthquake, the damper remains off during the strong motion part at the beginning of the ground motion as shown in Figure 6.15 and is activated at the far end (near 11.2s) which is useful for reducing the free vibration response of the structure. Overall, it was observed that the proposed control system works like a passive-on isolation system under long-period ground motions, while it performs similar to the passive-off system under short-period earthquakes.
Figure 6.14 Base floor acceleration ($a_{base}$) response under the 1979 Imperial Valley earthquake for the isolated building with the proposed controller

Figure 6.15 Base floor acceleration ($a_{base}$) response under the 1976 Friuli, Italy earthquake for the isolated building with the proposed controller

The hysteresis loops of the isolation system, i.e. the combined rubber bearing, MR damper, and linear bearings, are compared for all three control cases in Figure 6.16 under the 1979 Imperial Valley earthquake (long-period) and the 1976 Friuli, Italy earthquake
(short-period). It can be observed that the hysteresis loop of the proposed isolation system is very similar to that of the passive-on system and passive-off system under the long- and short-period earthquakes, respectively.

Figure 6.16 Hysteresis loops of the isolation system for three control cases: (a) under the 1979 Imperial Valley earthquake (long-period), (b) under the 1976 Friuli, Italy earthquake (short-period)

As explained earlier, the time delay and amplitude changes between the measured and target displacements are the main challenges in RTHS, which can impair the stability of the system. In order to evaluate the accuracy of the RTHSs, the time histories of the measured and target displacements along with the synchronization subspace plot [131] are plotted in Figure 6.17 under the 1976 Friuli earthquake. It can be clearly observed from Figure 6.17(a) that the measured displacement controlled by the ATS compensator has a
good agreement with the target displacement. In addition, the small amount of the Normalized Root Mean Square (NRMS) error provided at the top-left corner of Figure 6.17(b) shows a good actuator displacement control during the real-time tests. The NRMS error \( E^{NRMS} \) is determined as:

\[
E^{NRMS} = \frac{\sqrt{\sum^{N}_{i} (x^t_i - x^m_i)^2}}{\sqrt{\sum^{N}_{i} (x^t_i)^2}}
\]  

(6.6)

where \( x^t_i \) and \( x^m_i \) are the target and measured displacements, respectively. \( N \) is the number of data samples of displacement in the RTHS. If \( E^{NRMS} = 0 \), the target displacement and measured displacement at each time step would be the same and the synchronization subspace plot would be a perfectly diagonal straight line.

![Figure 6.17 Displacement control performance of the ATS compensator during RTHS under the 1976 Friuli earthquake: (a) overall displacement tracking, (b) synchronization subspace plot](image)

The time-history responses of the base-isolated structure under the other earthquake ground motions of Tables 6.4 and 6.5 are provided in Section A of Appendix B.
6.9 Comparison of RTHS Results with Numerical Simulation Results

The comparison of the RTHS results and numerical simulation results are presented in this section, where the experimental substructure is replaced by its numerical models (i.e., the MNS, Bouc-Wen and Colomb’s friction models for the MR damper, rubber bearing and linear bearings, respectively). Figures 6.18 compares the numerical simulation results with the RTHS results of the structure under the 1994 Northridge Sylmar long-period earthquake for the proposed system in the semi-active mode. The comparison of the RTHS and numerical simulation results for other earthquake ground motions are provided in section B of Appendix B.

Good agreement between the RTHS and numerical results was observed under most of the earthquake ground motions; however, the difference between the results arises once the superstructure is subjected to earthquakes with higher frequency contents (i.e., short-period motions). This can be seen in Figures AB.40 to AB.42 where the response of the structure under short-period motions is provided. As explained earlier, the direction of motion under short-period ground motions changes more frequently with a higher rate than under long-period ground motions. In addition to the inherent high frequencies of short-period excitations, the frequency contents of the motion are tripled herein due to the time scaling of the earthquake ground motions (i.e., scale factor = 1/3). However, the calibration of the numerical model parameters was conducted with a low frequency, making it harder for the numerical model to accurately estimate the actual behavior of the experimental base isolation system. Furthermore, the tiny gaps in the connection of the MR damper at both ends to the base isolator and the reaction frame provide an additional nonlinearity, which can provide additional noise-like accelerations to $a_{\text{base}}$, making the command signal of the
TSA controller different from that in the numerical simulation. This is the reason why the hysteresis loops of Figures AB.40 to AB.42 have differences between the numerical and RTHS results.

Although there are some differences between the numerical and experimental results, overall, the numerical simulation can predict the response of the base isolation system. It is quite a challenging task to accurately predict the highly nonlinear response of the base isolation system; this is why we need to conduct experimental tests to better understand the actual behavior of structures under dynamic loads [132].

![Comparison of the RTHS and numerical simulation results of the proposed system under the 1994 Northridge Sylmar long-period earthquake](image)

Figure 6.18 Comparison of the RTHS and numerical simulation results of the proposed system under the 1994 Northridge Sylmar long-period earthquake: (a) displacement response, (b) force response, (c) force-displacement relationship

### 6.10 Comparison of Statistical Responses

As stated previously, RTHS can provide statistical results as long as the test specimen does not have any significant failure under various repeated earthquake loadings.
Figure 6.19 shows the averaged values for the maximum structural responses of the isolated building with three different control cases under the short-period and scaled-down long-period earthquake ground motions of Tables 6.4 and 6.5. In Figure 6.19(a), the ideal effect of using high damping in reducing the base isolator displacement (from 24.58mm for the passive-off system to 7.26mm for passive-on) and third-floor velocity (from 161.2 mm/s for the passive-off system to 112.7 mm/s for passive-on system) can be clearly observed under long-period earthquake ground motions. The story drift of the passive-on system is slightly higher than the passive-off system due to the increase in the stiffness of the system with high damping. Also, no significant difference in the acceleration responses of the passive-on and passive-off systems is observed.

![Graphs showing structural responses](image)

Figure 6.19 Average of maximum structural responses of base-isolated three-story building: (a) under long-period ground motions, (b) under short-period ground motions
It can be clearly observed that high damping is beneficial in reducing the isolator displacement under long-period earthquake ground motion, as shown in Figure 6.19(a). Low damping is observed to be effective in reducing the story drift (reduced by about 73%) and floor acceleration (reduced by about 12% from 0.49g to 0.43g) under short-period ground motions. These results are similar to the observation by Providakis [9], where he evaluated the effect of supplemental damping on isolation systems subjected to far-fault (short-period) ground motions through numerical analysis, and concluded that additional damping increases the inter-story drifts and absolute floor accelerations under short-period earthquakes, while being effective on long-period earthquakes.

It can be observed from Figure 6.19 that the performance of the proposed TSA control law in the base isolation system is very similar to the passive-on isolation system under long-period earthquake ground motions and the passive-off system under short-period earthquake ground motions, which can increase the structural resiliency.

6.11 Summary

In this chapter, the performance of the proposed TSA control algorithm introduced in Chapter 3 was evaluated by conducting RTHSs. The base isolation system consisting of a small-scale rubber bearing combined with three linear bearings and a small-scale MR damper was used for the experimental substructure in the RTHS of this chapter. The superstructure above the isolation system was modelled numerically, consisting of the numerical substructure. A small-scale base-isolated structure was designed by implementing the simplified design procedure developed in Chapter 4. Characterization tests were conducted to identify the physical characteristics of the rubber bearing combined with the linear bearings and the MR damper. With the characterization tests, the parameters
for the MNS model, Bouc-Wen model, and Columb’s frictional model were identified to describe the dynamic response of the MR damper, rubber bearing, and linear bearings, respectively. Various long- and short-period earthquake ground motions were used to experimentally evaluate the performance of the three different base isolation systems. It was shown that the proposed isolation system achieves the desirable performance under long-period earthquake ground motions by increasing damping, while maintaining the unique performance of the conventional base isolation system under short-period earthquakes by decreasing damping.
CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 General

In this dissertation, a new semi-active base-isolation system was proposed to achieve structural resiliency under both short- and long-period earthquake ground motions. This study includes the development of an adaptive transmissibility-based semi-active (TSA) control algorithm, development of a simplified design procedure for the design of base-isolated structures with semi-active damping devices, the numerical assessment of the proposed algorithms and procedures, and experimental verifications by conducting real-time hybrid simulations. A brief description of the materials provided in each chapter as well as the corresponding conclusion is presented as follows.

7.2 Summary and Conclusions

In Chapter 2, previous research related to base isolation systems, different models for base isolators and MR dampers, and semi-active control algorithms implemented for base isolation system were reviewed. Detailed information for the MNS MR damper model and the Bouc-Wen base isolator model were provided. The implementation of various semi-active controllers for base isolation systems with MR dampers was illustrated, including the Lyapunov stability based controllers, linear quadratic regulator/Gaussian controllers, sliding mode control, fuzzy controllers, and neural network controllers. The practical application of these controllers is quite limited since they often work based on the control objectives that are not directly related to minimizing the maximum structural response of interest. Actually, some of these controllers (e.g., the linear quadratic regulator and sliding mode controller) work based on minimizing a quadratic cost function over the
entire control time which does not always lead to the minimization of the maximum structural response. In addition, the intelligent controllers (i.e. neural network and fuzzy controllers) require a nonlinear system to be optimized based on a training set or fuzzy logics. Due to the dependency of intelligent controllers on the selection of a pre-defined data set, their performance might be impaired for other inputs that have different characteristics from the pre-defined data set. Furthermore, the quantitative prediction of expected structural responses using these controllers is also difficult due to the complexity of the nonlinear system. To overcome these shortcomings, a new transmissibility based semi-active (TSA) controller was introduced in Chapter 3.

Chapter 3 developed a new control law in order to improve the performance of base-isolated structures with semi-active damping devices under earthquake ground motions with various frequency contents. The transmissibility theory of a single-degree-of-freedom (SDOF) structure subjected to a steady-state response under harmonic loading was considered as the basis for the TSA controller. According to the transmissibility theory, it was shown that high damping is beneficial under long-period excitations, while low damping is effective under short-period excitations. The TSA control law can adaptively change the damping depending on the frequency contents in the response of the base-isolated structure by achieving maximum damping under long-period excitations and minimum damping under short-period.

The TSA controller requires local feedback data of the structural acceleration response which makes this system cost-effective in comparison to most of the controllers that require full-state feedback data of the structural response. Furthermore, unlike existing methods, the TSA control law enables the response prediction of base-isolated structures.
with semi-active damping devices without implementing any complex nonlinear time history analysis, which can be incorporated into a practical design procedure. This procedure was developed in Chapter 4 and named the simplified design procedure.

Based on the abilities provided by the TSA control law in Chapter 3, a simplified design procedure for estimating the response of the base-isolated structures with MR dampers was developed in Chapter 4. This procedure enabled the design of base-isolated structures with MR dampers without conducting nonlinear time history analysis. The simplified design procedure works based on the effective period and equivalent damping ratio to find the maximum displacement and maximum base shear of the isolation system using the design spectrum. The equations describing the energy dissipation and effective stiffness of the base isolators and MR dampers were presented using the bilinear model and the Herschel-Bulkley model, respectively. Actually, by determining the parameters of these models, the response estimation of the base-isolated structure was feasible.

The performance of the TSA controller and the accuracy of the simplified design procedure were evaluated numerically in Chapter 5 for a full-scale base-isolated three-story building with MR dampers. Experimental evaluation of the proposed methods was presented in Chapter 6 through conducting real-time hybrid simulations for a small-scale base-isolated structure with MR dampers.

Chapter 5 numerically evaluated the performance of the proposed base-isolation system. A base-isolated three-story building with MR dampers was designed by implementing the simplified design procedure developed in Chapter 4. The performance of the proposed TSA control law was validated by conducting nonlinear time history analyses (NTHA) with selected long- and short-period earthquake ground motions. The
Bouc-Wen hysteresis model for base isolators and the MNS model for MR dampers were used in the numerical simulations. The parameters for these numerical models were determined based on the results from the simplified design procedure to achieve the maximum equivalent damping ratio of $\zeta_{eq}=40\%$ and the minimum equivalent damping ratio of $\zeta_{eq}=10\%$ by controlling the command current to the damper. The structural responses of the proposed system were compared with the system with maximum and minimum equivalent damping ratios (i.e., passive-on and passive-off, respectively). Detailed information about the performance of the controller on sending a command signal under different earthquake ground motions was provided.

According to the numerical simulation results, it was clearly evident that the use of high damping is effective in reducing the structural responses, especially the isolator displacement, under long-period earthquake ground motions. However, in order to protect nonstructural components, the implementation of low damping would be very effective especially under short-period earthquake ground motions. Actually, this shows that satisfying the high-level seismic performance objectives would be very difficult by just implementing passive isolation systems. Unlike these passive isolation systems, it was demonstrated that the proposed isolation system can resolve this issue; the proposed TSA control law makes the building work like a passive-on or passive-off isolation system as necessary to achieve high performance level under both long- and short-period earthquake ground motions, which can significantly improve the resiliency and sustainability of buildings.

In Chapter 6, a detailed description of the real-time hybrid simulation tests that were conducted to investigate the performance of the proposed TSA controller was provided.
The hybrid simulations was divided into experimental and numerical substructures representing a small-scale three-story base-isolated building with MR dampers. The experimental substructure consisted of a small-scale rubber bearing combined with three linear bearings and a small-scale MR damper. The superstructure above the isolation system was considered as the numerical substructure. The small-scale base-isolated building was designed by implementing the simplified design procedure developed in Chapter 4 to accommodate the restrictions given in the experimental substructure in terms of the displacement amplitude and the force capacity of the servo-hydraulic actuator system.

Characterization tests were conducted on the rubber bearing combined with the linear bearings and the MR damper to evaluate their performance and to identify the parameters of their numerical models, where the MNS model, Bouc-Wen model, and Coulomb’s friction model were used for the MR damper, rubber bearing, and linear bearings, respectively. Various long- and short-period earthquake ground motions were applied to the experimental substructure and the statistical analysis of the maximum structural responses was provided. The performance of the isolated building controlled with the passive-off, passive-on, and proposed methods was experimentally investigated, where the experimental results were compared with the numerical simulation results. Statistical responses of all hybrid simulations were provided as well. It was clearly shown that the proposed isolation system achieves the desirable performance under long-period earthquake ground motions by increasing damping, while maintaining the unique performance of the conventional base isolation system under short-period earthquake ground motions by decreasing damping.
7.3 Future Research

Further studies on the following topics are recommended:

In this study, the performance of the TSA controller was only experimentally evaluated under the five long-period and another five short-period earthquake ground motions. It is recommended to further evaluate the performance of the TSA controller experimentally and determine how the TSA controller satisfies the given performance objectives under various earthquake ground motions scaled to the DBE and MCE levels.

Further investigations on the parameters of the TSA controller and their effects on the response of the semi-actively controlled base-isolated structures need to be conducted, where various earthquake ground motions should be implemented and the effects of noise and possible errors should be evaluated.

The simplified design procedure in this study was used to design one full-scale and one small-scale three-story base-isolated buildings. It is recommended that various base-isolated structures with MR dampers be designed by implementing this procedure. Various building design parameters such as building geometry, height, plan, mass distribution, etc. need to be considered in the selection of the base isolated buildings, and the response estimation from the simplified design procedure should be assessed both numerically and experimentally to further improve the capability of the simplified design procedure.

The behavior of superstructures above the isolation layers was assumed to be linear elastic in this dissertation. It is suggested to extend the performance evaluation of the proposed TSA controller by further considering the nonlinear response of superstructures.

Due to the limitations of the laboratory and equipment, a small-scale base-isolation system was considered in the experimental test of this dissertation. It is strongly
recommended to implement large-scale experimental tests with full-scale base isolators and semi-active damping devices to further investigate the performance of the proposed base isolation system. This will ensure the practical application of the proposed base isolation system to increase the resiliency and sustainability of civil infrastructure.
BIBLIOGRAPHY


APPENDIX A

NUMERICAL SIMULATION RESULTS UNDER VARIOUS EARTHQUAKE GROUND MOTIONS

Figure AA.1 Responses of the isolated building under the 1994 Northridge, Sylmar earthquake (long-period): (a) ground acceleration, (b) base isolator displacement, (c) 3\textsuperscript{rd} story drift (d) 3\textsuperscript{rd} floor absolute velocity, and (e) 3\textsuperscript{rd} floor absolute acceleration.
Figure AA.2 Base floor acceleration ($a_{base}$) response of the isolated building with the proposed method under the 1994 Northridge Sylmar Converter (long-period)

Figure AA.3 Comparison of hysteresis loops of the base isolator and the MR damper for each control case under the 1994 Northridge Sylmar Converter (long-period)
Figure AA.4 Responses of the isolated building under the 1999 Chi-Chi, Taiwan earthquake (long-period): (a) ground acceleration, (b) base isolator displacement, (c) 3rd story drift (d) 3rd floor absolute velocity, and (e) 3rd floor absolute acceleration.
Figure AA.5 Base floor acceleration ($a_{\text{base}}$) response of the isolated building with the proposed method under the 1999 Chi-Chi, Taiwan (long-period)

Figure AA.6 Comparison of hysteresis loops of the base isolator and the MR damper for each control case under the 1999 Chi-Chi, Taiwan (long-period)
Figure AA.7 Responses of the isolated building under the 1994 Northridge, Jensen earthquake (long-period): (a) ground acceleration, (b) base isolator displacement, (c) 3rd story drift (d) 3rd floor absolute velocity, and (e) 3rd floor absolute acceleration
Figure AA.8 Base floor acceleration ($a_{\text{base}}$) response of the isolated building with the proposed method under the 1994 Northridge, Jensen (long-period)

Figure AA.9 Comparison of hysteresis loops of the base isolator and the MR damper for each control case under the 1994 Northridge, Jensen (long-period)
Figure AA.10 Responses of the isolated building under the 1990 Manjil, Iran earthquake (long-period): (a) ground acceleration, (b) base isolator displacement, (c) 3rd story drift, (d) 3rd floor absolute velocity, and (e) 3rd floor absolute acceleration.
Figure AA.11 Base floor acceleration ($a_{base}$) response of the isolated building with the proposed method under the 1990 Manjil, Iran (long-period)

Figure AA.12 Comparison of hysteresis loops of the base isolator and the MR damper for each control case under the 1990 Manjil, Iran (long-period)
Figure AA.13 Responses of the isolated building under the 1994 Northridge, Canyon County earthquake (short-period): (a) ground acceleration, (b) base isolator displacement, (c) 3rd story drift (d) 3rd floor absolute velocity, and (e) 3rd floor absolute acceleration.
Figure AA.14 Base floor acceleration ($a_{\text{base}}$) response of the isolated building with the proposed method under the 1994 Northridge, Canyon County (short-period)

Figure AA.15 Comparison of hysteresis loops of the base isolator and the MR damper for each control case under the 1994 Northridge, Canyon County (short-period)
Figure AA.16 Responses of the isolated building under the 1992 Landers, Coolwater earthquake (short-period): (a) ground acceleration, (b) base isolator displacement, (c) 3\textsuperscript{rd} story drift (d) 3\textsuperscript{rd} floor absolute velocity, and (e) 3\textsuperscript{rd} floor absolute acceleration
Figure AA.17 Base floor acceleration ($a_{base}$) response of the isolated building with the proposed method under the 1992 Landers, Cool-water (short-period)

Figure AA.18 Comparison of hysteresis loops of the base isolator and the MR damper for each control case under the 1992 Landers, Cool-water (short-period)
Figure AA.19 Responses of the isolated building under the 1994 Northridge, Beverly Hills earthquake (short-period): (a) ground acceleration, (b) base isolator displacement, (c) 3rd story drift (d) 3rd floor absolute velocity, and (e) 3rd floor absolute acceleration
Figure AA.20 Base floor acceleration \( (a_{base}) \) response of the isolated building with the proposed method under the 1994 Northridge, Beverly Hills (short-period)

Figure AA.21 Comparison of hysteresis loops of the base isolator and the MR damper for each control case under the 1994 Northridge, Beverly Hills (short-period)
Figure AA.22 Responses of the isolated building under the 1999 Kocaeli, Turkey earthquake (short-period): (a) ground acceleration, (b) base isolator displacement, (c) 3rd story drift (d) 3rd floor absolute velocity, and (e) 3rd floor absolute acceleration
Figure AA.23 Base floor acceleration ($a_{base}$) response of the isolated building with the proposed method under the 1999 Kocaeli, Turkey (short-period)

Figure AA.24 Comparison of hysteresis loops of the base isolator and the MR damper for each control case under the 1999 Kocaeli, Turkey (short-period)
APPENDIX B

REAL-TIME HYBRID SIMULATION RESULTS UNDER VARIOUS EARTHQUAKE GROUND MOTIONS

Section A:

Figure AB.1 Displacement control using the ATS compensator under the 1979 Imperial Valley earthquake (long-period): (a) overall displacement tracking, (b) synchronization subspace plot
Figure AB.2 Time history responses under the 1994 Northridge Sylmar Convertor earthquake (long-period) for the isolated building: (a) ground acceleration, (b) base isolator displacement, (c) 3rd story drift, (d) 3rd floor absolute velocity, and (e) 3rd floor absolute acceleration
Figure AB.3 Base floor acceleration ($a_{\text{base}}$) response under the 1994 Northridge Sylmar Convertor earthquake (long-period) for the isolated building with the proposed controller.

Figure AB.4 Hysteresis loops of the isolation system for three control cases under the 1994 Northridge Sylmar Convertor earthquake (long-period).
Figure AB.5 Displacement control using the ATS compensator under the 1994 Northridge Sylmar Convertor (long-period): (a) overall displacement tracking, (b) synchronization subspace plot.
Figure AB.6 Time history responses under the 1999 Chi-Chi, Taiwan earthquake (long-period) for the isolated building: (a) ground acceleration, (b) base isolator displacement, (c) 3rd story drift, (d) 3rd floor absolute velocity, and (e) 3rd floor absolute acceleration
Figure AB.7 Base floor acceleration ($a_{\text{base}}$) response under the 1999 Chi-Chi, Taiwan earthquake (long-period) for the isolated building with the proposed controller.

Figure AB.8 Hysteresis loops of the isolation system for three control cases under the 1999 Chi-Chi, Taiwan earthquake (long-period).
Figure AB.9 Displacement control using the ATS compensator under the 1999 Chi-Chi, Taiwan (long-period): (a) overall displacement tracking, (b) synchronization subspace plot.
Figure AB.10 Time history responses under the 1994 Northridge, Jensen earthquake (long-period) for the isolated building: (a) ground acceleration, (b) base isolator displacement, (c) 3rd story drift, (d) 3rd floor absolute velocity, and (e) 3rd floor absolute acceleration
Figure AB.11 Base floor acceleration ($a_{base}$) response under the 1994 Northridge, Jensen earthquake (long-period) for the isolated building with the proposed controller

Figure AB.12 Hysteresis loops of the isolation system for three control cases under the 1994 Northridge, Jensen earthquake (long-period)
Figure AB.13 Displacement control using the ATS compensator under the 1994 Northridge, Jensen (long-period): (a) overall displacement tracking, (b) synchronization subspace plot
Figure AB.14 Time history responses under the 1990 Manjil, Iran earthquake (long-period) for the isolated building: (a) ground acceleration, (b) base isolator displacement, (c) 3rd story drift, (d) 3rd floor absolute velocity, and (e) 3rd floor absolute acceleration.
Figure AB.15 Base floor acceleration ($a_{\text{base}}$) response under the 1990 Manjil, Iran earthquake (long-period) for the isolated building with the proposed controller.

Figure AB.16 Hysteresis loops of the isolation system for three control cases under the 1990 Manjil, Iran earthquake (long-period).
Figure AB.17 Displacement control using the ATS compensator under the 1990 Manjil, Iran (long-period): (a) overall displacement tracking, (b) synchronization subspace plot.
Figure AB.18 Time history responses under the 1989 Loma Prieta, Capitola earthquake (short-period) for the isolated building: (a) ground acceleration, (b) base isolator displacement, (c) 3rd story drift, (d) 3rd floor absolute velocity, and (e) 3rd floor absolute acceleration.
Figure AB.19 Base floor acceleration ($a_{base}$) response under the 1989 Loma Prieta, Capitola earthquake (short-period) for the isolated building with the proposed controller.

Figure AB.20 Hysteresis loops of the isolation system for three control cases under the 1989 Loma Prieta, Capitola earthquake (short-period).
Figure AB.21 Displacement control using the ATS compensator under the 1989 Loma Prieta, Capitola earthquake (short-period): (a) overall displacement tracking, (b) synchronization subspace plot.
Figure AB.22 Time history responses under the 1971 San Fernando, LA Hollywood earthquake (short-period) for the isolated building: (a) ground acceleration, (b) base isolator displacement, (c) 3rd story drift, (d) 3rd floor absolute velocity, and (e) 3rd floor absolute acceleration
Figure AB.23 Base floor acceleration ($a_{\text{base}}$) response under the 1971 San Fernando, LA Hollywood earthquake (short-period) for the isolated building with the proposed controller.

Figure AB.24 Hysteresis loops of the isolation system for three control cases under the 1971 San Fernando, LA Hollywood earthquake (short-period).
Figure AB.25 Displacement control using the ATS compensator under the 1971 San Fernando, LA Hollywood earthquake (short-period): (a) overall displacement tracking, (b) synchronization subspace plot.
Figure AB.26 Time history responses under the 1992 Cape Mendocino, Rio Del Overpass earthquake (short-period) for the isolated building: (a) ground acceleration, (b) base isolator displacement, (c) 3rd story drift, (d) 3rd floor absolute velocity, and (e) 3rd floor absolute acceleration.
Figure AB.27 Base floor acceleration ($a_{\text{base}}$) response under the 1992 Cape Mendocino, Rio Del Overpass earthquake (short-period) for the isolated building with the proposed controller.

Figure AB.28 Hysteresis loops of the isolation system for three control cases under the 1992 Cape Mendocino, Rio Del Overpass earthquake (short-period).
Figure AB.29 Displacement control using the ATS compensator under the 1992 Cape Mendocino, Rio Del Overpass earthquake (short-period): (a) overall displacement tracking, (b) synchronization subspace plot

NRMS error = 0.0293
Figure AB.30 Time history responses under the 1994 Northridge, Canyon County earthquake (short-period) for the isolated building: (a) ground acceleration, (b) base isolator displacement, (c) 3rd story drift, (d) 3rd floor absolute velocity, and (e) 3rd floor absolute acceleration
Figure AB.31 Base floor acceleration ($a_{base}$) response under the 1994 Northridge, Canyon County earthquake (short-period) for the isolated building with the proposed controller.

Figure AB.32 Hysteresis loops of the isolation system for three control cases under the 1994 Northridge, Canyon County earthquake (short-period).
Figure AB.33 Displacement control using the ATS compensator under the 1994 Northridge, Canyon County earthquake (short-period): (a) overall displacement tracking, (b) synchronization subspace plot
Section B:

Figure AB.34 Comparison of the RTHS and numerical simulation results of proposed system under the 1990 Manjil, Iran (long-period): (a) displacement response, (b) force response, (c) force-displacement relationship

Figure AB.35 Comparison of the RTHS and numerical simulation results of proposed system under the 1994 Northridge, Jensen (long-period): (a) displacement response, (b) force response, (c) force-displacement relationship
Figure AB.36 Comparison of the RTHS and numerical simulation results of proposed system under the 1999 Chi-Chi, Taiwan earthquake (long-period): (a) displacement response, (b) force response, (c) force-displacement relationship

Figure AB.37 Comparison of the RTHS and numerical simulation results of proposed system under the 1979 Imperial Valley earthquake (long-period): (a) displacement response, (b) force response, (c) force-displacement relationship
Figure AB.38 Comparison of the RTHS and numerical simulation results of proposed system under the 1989 Loma Prieta, Capitola earthquake (short-period): (a) displacement response, (b) force response, (c) force-displacement relationship

Figure AB.39 Comparison of the RTHS and numerical simulation results of proposed system under the 1976 Friuli, Italy earthquake (short-period): (a) displacement response, (b) force response, (c) force-displacement relationship
Figure AB.40 Comparison of the RTHS and numerical simulation results of proposed system under the 1992 Cape Mendocino, Rio Del Overpass earthquake (short-period): (a) displacement response, (b) force response, (c) force-displacement relationship

Figure AB.41 Comparison of the RTHS and numerical simulation results of proposed system under the 1994 Northridge, Canyon County (short-period): (a) displacement response, (b) force response, (c) force-displacement relationship
Figure AB.42 Comparison of the RTHS and numerical simulation results of proposed system under the 1971 San Fernando, LA Hollywood earthquake (short-period): (a) displacement response, (b) force response, (c) force-displacement relationship
VITA

Ramin Rabiee was born on August 25, 1988 in Isfahan, Iran. He received his Bachelor of Science degree in Civil Engineering in August 2010 from Azad University, Isfahan, Iran. Upon graduation, he started to work as a site engineer for Bana Gostar Armand Company, Isfahan, Iran, where he was responsible for supervising a road construction site project. He earned a Master of Science degree in Earthquake Engineering in July 2013 from Shiraz University of Technology, Shiraz, Iran. The title of his master’s thesis is “Evaluation of Dynamic Characteristics and Vulnerability Index of Historical Buildings in Iran Using Microtremor Measurements”. He worked as a site engineer and the main contractor for the construction of several residential buildings with Ara Se Ram Rasta Company, Isfahan, Iran. In fall of 2014, he began to pursue his Ph.D. studies in Structural Engineering at Old Dominion University. His dissertation research focus was on the development of an adaptive semi-active base-isolation system to achieve structural resiliency under various earthquake ground motions. During this period, he also collaborated on several different numerical and experimental projects related to effective force testing for nonlinear structures and force control of servo-hydraulic actuator systems in real-time hybrid simulations. He has authored and co-authored several papers in top peer-reviewed scientific journals. Ramin received the Engineering Dean’s Graduate Fellowship Award for Excellence from the Batten College of Engineering and Technology of Old Dominion University for the academic year of 2015-2016. He was also selected as the Excellent Research Assistant Award winner of 2019 by the faculty members of the Civil and Environmental Engineering Department of Old Dominion University.