Shake Table Tests of a Two-Story Woodframe House

David Fischer
André Filiatrault
Bryan Folz
Chia-Ming Uang
Frieder Seible

Department of Structural Engineering
University of California, San Diego

2001
The CUREE-Caltech Woodframe Project is funded by the Federal Emergency Management Agency (FEMA) through a Hazard Mitigation Grant Program award administered by the California Governor’s Office of Emergency Services (OES) and is supported by non-Federal sources from industry, academia, and state and local government. California Institute of Technology (Caltech) is the prime contractor to OES. The Consortium of Universities for Research in Earthquake Engineering (CUREE) organizes and carries out under subcontract to Caltech the tasks involving other universities, practicing engineers, and industry.

Disclaimer

The information in this publication is presented as a public service by California Institute of Technology and the Consortium of Universities for Research in Earthquake Engineering. No liability for the accuracy or adequacy of this information is assumed by them, nor by the Federal Emergency Management Agency and the California Governor’s Office of Emergency Services, which provide funding for this project.
Shake Table Tests of a Two-Story Woodframe House

David Fischer
André Filiatrault
Bryan Folz
Chia-Ming Uang
Frieder Seible

Department of Structural Engineering
University of California, San Diego

2001
ACKNOWLEDGEMENTS

The research project described in this report was funded by the California Universities for Earthquake Engineering (CUREe) as part of the CUREe-Caltech Woodframe Project (“Earthquake Hazard Mitigation of Woodframe Construction”), under a grant administered by the California Office of Emergency Services and funded by the Federal Emergency Management Agency. The following organizations are also acknowledged for providing financial and in-kind support to Element 1 – Testing and Analysis of the CUREe-Caltech Woodframe Project: Ainsworth Lumber Co., American Plywood Association (APA), Dixieline Lumber Co., International Staples, Nails and Tools Association (ISANTA), Johns-Manville Roofing Materials, Maruhachi Ceramics of America Inc., Structural Engineering Association of Northern California (SEAONC), Simpson Strong Tie Inc., Stimson Lumber Co., Valentine Construction Inc., Western Forest Product Association (WFPA), Willamette Industry, and Windowmaster Products.

We greatly appreciated the input and coordination provided by Professor John Hall of the California Institute of Technology and by Mr. Robert Reitherman of the California Universities for Research in Earthquake Engineering.
## LIST OF SYMBOLS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a(\omega)$</td>
<td>Real part of frequency response function from fast Fourier transform of ambient vibration test data</td>
</tr>
<tr>
<td>$A_{chord}$</td>
<td>Area of chord member of floor diaphragm</td>
</tr>
<tr>
<td>$A_i$</td>
<td>Modal value corresponding to channel $i$</td>
</tr>
<tr>
<td>$A_{OAB}$</td>
<td>Area of triangle OAB used in computing equivalent viscous damping</td>
</tr>
<tr>
<td>$b$</td>
<td>Width of pair of diagonal deformation instruments on floor diaphragm</td>
</tr>
<tr>
<td>$b(\omega)$</td>
<td>Imaginary part of frequency response function from fast Fourier transform of ambient vibration test data</td>
</tr>
<tr>
<td>$\bar{c}$</td>
<td>Damping constant of Kelvin solid viscoelastic element</td>
</tr>
<tr>
<td>$d$</td>
<td>Depth of floor diaphragm (16 feet)</td>
</tr>
<tr>
<td>$E$</td>
<td>Elastic modulus of wood framing member</td>
</tr>
<tr>
<td>$E_d$</td>
<td>Energy dissipated per cycle in Kelvin solid viscoelastic element</td>
</tr>
<tr>
<td>$e_n$</td>
<td>Nail deformation constant used in computing nail slip deflection</td>
</tr>
<tr>
<td>$EI$</td>
<td>Flexural stiffness of floor diaphragm from quasi-static tests</td>
</tr>
<tr>
<td>$F$</td>
<td>Force applied to floor diaphragm during quasi-static tests</td>
</tr>
<tr>
<td>$F_{max}$</td>
<td>Maximum force applied to floor diaphragm during quasi-static tests</td>
</tr>
<tr>
<td>$F_{min}$</td>
<td>Minimum force applied to floor diaphragm during quasi-static tests</td>
</tr>
<tr>
<td>$F_i$</td>
<td>Intercept force of shearwall for Wayne Stewart hysteresis</td>
</tr>
<tr>
<td>$F_u$</td>
<td>Ultimate force of shearwall for Wayne Stewart hysteresis</td>
</tr>
<tr>
<td>$F_y$</td>
<td>Yield force of shearwall for Wayne Stewart hysteresis</td>
</tr>
<tr>
<td>$f_{Nyquist}$</td>
<td>Nyquist frequency during ambient vibration tests</td>
</tr>
<tr>
<td>$F(\omega)$</td>
<td>Frequency response function from fast Fourier transform of ambient vibration test data</td>
</tr>
<tr>
<td>$G$</td>
<td>Shear modulus of sheathing (floor or wall)</td>
</tr>
<tr>
<td>$GA_s$</td>
<td>Shear stiffness of floor diaphragm from quasi-static tests</td>
</tr>
<tr>
<td>$I$</td>
<td>Moment of inertia of chord member of floor diaphragm</td>
</tr>
<tr>
<td>$k_o$</td>
<td>Initial lateral stiffness of equivalent SDOF system</td>
</tr>
<tr>
<td>$k_{flexural}$</td>
<td>Secant flexural stiffness of floor diaphragm during quasi-static tests</td>
</tr>
<tr>
<td></td>
<td>(shear force divided by flexural deformation)</td>
</tr>
<tr>
<td>$k_n$</td>
<td>Lateral stiffness of equivalent SDOF system following seismic test</td>
</tr>
<tr>
<td></td>
<td>Level $n$</td>
</tr>
</tbody>
</table>
$k_{shear}$  Secant shear stiffness of floor diaphragm during quasi-static tests (shear force divided by shear deformation)

$L$  Length of floor diaphragm (20 feet)

$m$  Mass of mass-spring system

$MC$  Moisture content of wood samples in percent

$N$  Number of data points per window from ambient vibration test data

$N_w$  Number of windows for time domain ambient vibration test data

$P_{TRI}$  Post-ultimate stiffness factor for Wayne Stewart hysteresis

$P_{UNL}$  Unloading stiffness factor for Wayne Stewart hysteresis

$R$  Constant representing initial conditions for a single-degree-of-freedom system undergoing viscously damped free vibration

$R_f$  Post-yield stiffness factor for Wayne Stewart hysteresis

$s_o$  Time duration of each window from ambient vibration test data

$S_{total}$  Total time duration of all windows from ambient vibration test data

$t$  Time

$T_d$  Damped natural period of a system

$u(t)$  Displacement of a single-degree-of-freedom system undergoing viscously damped free vibration

$\dot{u}(t)$  Velocity of a single-degree-of-freedom system undergoing viscously damped free vibration

$\ddot{u}(t)$  Acceleration of a single-degree-of-freedom system undergoing viscously damped free vibration

$V$  Shear force applied to floor diaphragm during quasi-static tests

$V_{max}$  Maximum shear force applied to floor diaphragm during quasi-static tests

$V_{min}$  Minimum shear force applied to floor diaphragm during quasi-static tests

$X_o$  Maximum displacement of Kelvin solid viscoelastic element

$\alpha$  Peak spectral density amplitude for a reference channel in computing mode shape

$\alpha_1$  Reloading stiffness factor for Wayne Stewart hysteresis

$\beta$  Peak spectral density amplitude for a particular channel

$\beta_1$  Softening factor for Wayne Stewart hysteresis

$\delta$  Logarithmic decrement

$\Delta f$  Frequency resolution of ambient vibration test data
$\Delta_{\text{flexural}}$  Flexural deformation of floor diaphragm

$\Delta_{\text{flexural, max}}$  Maximum flexural deformation of floor diaphragm

$\Delta_{\text{flexural, min}}$  Minimum flexural deformation of floor diaphragm

$\Delta_{\text{global}}$  Global deformation at center of floor diaphragm

$\Delta_{\text{global, max}}$  Maximum global deformation at center of floor diaphragm

$\Delta_{\text{global, min}}$  Minimum global deformation at center of floor diaphragm

$\Delta_{L}$  Average diagonal deformation from a pair of diagonal deformation instruments on floor diaphragm during quasi-static tests

$\Delta_{n}$  Deflection due to nail slip

$\Delta_{\text{shear}}$  Shear deformation of floor diaphragm

$\Delta_{\text{shear, max}}$  Maximum shear deformation of floor diaphragm

$\Delta_{\text{shear, min}}$  Minimum shear deformation of floor diaphragm

$\Delta_{\text{shearwall}}$  Average of east shearwall deflection and west shearwall deflection

$\Delta t$  Data sampling rate

$\Delta\bar{u}_i$  Peak-to-peak acceleration between peaks during cycle $i$ of free vibration

$\Delta\bar{u}_{\text{forced}}$  Peak-to-peak acceleration between peaks of forced vibration

$\Delta u_{\text{un}}$  Maximum displacement at current stage of hysteresis loop for Wayne Stewart hysteresis

$\gamma$  Shear strain in floor diaphragm

$\phi$  Constant representing initial conditions for a single-degree-of-freedom system undergoing viscously damped free vibration

$\phi(\omega)$  Phase angle as function of natural frequency

$\omega$  Natural frequency of mass-spring system

$\omega_0$  Initial natural frequency of test structure

$\omega_d$  Damped natural frequency of a system

$\omega_n$  Natural frequency of test structure following seismic test Level $n$

$\zeta$  Viscous damping ratio for a single-degree-of-freedom system undergoing viscously damped free vibration

$\zeta_{eq}$  Equivalent viscous damping ratio
# TABLE OF CONTENTS

DISCLAIMER ................................................................................................................. i

ACKNOWLEDGEMENTS ..................................................................................................... ii

LIST OF SYMBOLS ......................................................................................................... iii

TABLE OF CONTENTS .................................................................................................... vi

SCOPE OF RESEARCH .................................................................................................. ix

REPORT LAYOUT ............................................................................................................. x

<table>
<thead>
<tr>
<th>Chapter</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Introduction</td>
<td>1</td>
</tr>
<tr>
<td>1.1</td>
<td>Description of the CUREe-Caltech Woodframe Project</td>
<td>2</td>
</tr>
<tr>
<td>1.2</td>
<td>Description of Task 1.1.1: Shake Table Tests of a Simplified Two-Story Single Family House</td>
<td>5</td>
</tr>
<tr>
<td>2</td>
<td>Literature Review on Full-Scale Testing of Woodframe Structures</td>
<td>7</td>
</tr>
<tr>
<td>2.1</td>
<td>Review of Previous Studies</td>
<td>7</td>
</tr>
<tr>
<td>2.2</td>
<td>Motivation for Research</td>
<td>52</td>
</tr>
<tr>
<td>3</td>
<td>Test Structure and Testing Objectives</td>
<td>54</td>
</tr>
<tr>
<td>3.1</td>
<td>Description of Test Structure</td>
<td>54</td>
</tr>
<tr>
<td>3.2</td>
<td>UC San Diego Uniaxial Seismic Simulation Facility</td>
<td>57</td>
</tr>
<tr>
<td>3.3</td>
<td>Experimental Setup</td>
<td>59</td>
</tr>
<tr>
<td>3.4</td>
<td>Testing Objectives</td>
<td>61</td>
</tr>
<tr>
<td>4</td>
<td>Description of Shake Table Tests</td>
<td>64</td>
</tr>
<tr>
<td>4.1</td>
<td>Testing Sequences</td>
<td>65</td>
</tr>
<tr>
<td>4.2</td>
<td>Description of Test Phases 1 – 4</td>
<td>70</td>
</tr>
<tr>
<td>4.3</td>
<td>Description of Test Phase 5</td>
<td>73</td>
</tr>
<tr>
<td>4.4</td>
<td>Description of Test Phase 6</td>
<td>75</td>
</tr>
<tr>
<td>4.5</td>
<td>Description of Test Phase 6A</td>
<td>76</td>
</tr>
<tr>
<td>4.6</td>
<td>Description of Test Phase 7</td>
<td>77</td>
</tr>
<tr>
<td>4.7</td>
<td>Description of Test Phase 7A</td>
<td>80</td>
</tr>
<tr>
<td>4.8</td>
<td>Description of Test Phase 8</td>
<td>82</td>
</tr>
<tr>
<td>4.9</td>
<td>Description of Test Phase 9</td>
<td>84</td>
</tr>
<tr>
<td>4.10</td>
<td>Description of Test Phase 10</td>
<td>85</td>
</tr>
<tr>
<td>4.11</td>
<td>Supplemental Weight Distribution</td>
<td>88</td>
</tr>
<tr>
<td>5</td>
<td>Description of Instrumentation for Shake Table Tests</td>
<td>92</td>
</tr>
<tr>
<td>5.1</td>
<td>Types of Instrumentation for Shake Table Tests</td>
<td>93</td>
</tr>
<tr>
<td>5.2</td>
<td>Instrumentation for Test Phases 1 – 4</td>
<td>99</td>
</tr>
<tr>
<td>5.3</td>
<td>Instrumentation for Test Phase 5</td>
<td>99</td>
</tr>
<tr>
<td>5.4</td>
<td>Instrumentation for Test Phases 6 &amp; 6A</td>
<td>99</td>
</tr>
<tr>
<td>5.5</td>
<td>Instrumentation for Test Phase 7</td>
<td>100</td>
</tr>
<tr>
<td>5.6</td>
<td>Instrumentation for Test Phase 7A</td>
<td>100</td>
</tr>
<tr>
<td>5.7</td>
<td>Instrumentation for Test Phase 8</td>
<td>101</td>
</tr>
</tbody>
</table>
5.8 Instrumentation for Test Phase 9 ........................................................... 101
5.9 Instrumentation for Test Phase 10 ......................................................... 102

Chapter 6 Determination of Material Properties ................................................. 103
6.1 Moisture Content and Specific Gravity of Wood Materials .................. 103
6.2 Hysteretic Properties of Sheathing to Framing Lumber Connections ... 105
6.3 Compressive Strength of Exterior Stucco ............................................... 112

Chapter 7 Results of Quasi-Static Tests .......................................................... 114
7.1 Diaphragm Analysis ............................................................................. 115
7.2 Variation of Global Stiffness, Shear Stiffness, Flexural Stiffness, and Damping of Diaphragm ............................................................. 123
7.3 Diaphragm Flexibility ........................................................................... 126
7.4 Diaphragm Deflection Using Code Equations ...................................... 128
7.5 Phase Lag Between Shear and Global Diaphragm Deformation .......... 131
7.6 Conclusions and Recommendations .................................................... 135

Chapter 8 Results of Frequency Evaluation Tests ............................................ 137
8.1 Testing Procedure .................................................................................. 137
8.2 Data Analysis ....................................................................................... 140
8.3 Variation of Natural Frequency ............................................................. 143
8.4 Variation of Normalized Lateral Stiffness ............................................ 146
8.5 Variation of Mode Shapes ..................................................................... 149

Chapter 9 Results of Damping Evaluation Tests ........................................... 154
9.1 Testing Procedure ................................................................................ 154
9.2 Data Analysis ....................................................................................... 155
9.3 Damping Test Results .......................................................................... 159

Chapter 10 Results of Seismic Tests ............................................................. 162
10.1 Shake Table Fidelity ........................................................................... 163
10.2 Description of Test Results ................................................................. 163
10.3Selected Comparative Seismic Responses ........................................... 169
10.4 Summary of Seismic Test Results ....................................................... 226

Chapter 11 Numerical Modeling of Test Structure ........................................ 229
11.1Description of Pancake Model .............................................................. 230
11.2 Shearwall Hysteretic Parameters .......................................................... 235
11.3 Pushover Analysis ............................................................................. 238
11.4 Comparison of Fundamental Frequencies and Mode Shapes .......... 240
11.5 Comparison of Numerical Results with Experimental Results for Test Phase 9 ................................................................. 242

Chapter 12 Summary and Conclusions ....................................................... 249
12.1 Summary ............................................................................................. 249
12.2 Conclusions ....................................................................................... 250

Chapter 13 Appendices ............................................................................... 253
Appendix A Architectural and Structural Drawings of Test Structure .......... 254
Appendix B Description of Experimental Tests ......................................... 262
<table>
<thead>
<tr>
<th>Appendix</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>Instrumentation</td>
<td>275</td>
</tr>
<tr>
<td>D</td>
<td>Sheathing to Framing Lumber Connection Test Results</td>
<td>304</td>
</tr>
<tr>
<td>E</td>
<td>Selected Quasi-Static Results</td>
<td>310</td>
</tr>
<tr>
<td>F</td>
<td>Selected Frequency Test Results</td>
<td>325</td>
</tr>
<tr>
<td>G</td>
<td>Selected Damping Test Results</td>
<td>369</td>
</tr>
<tr>
<td>H</td>
<td>Selected Seismic Results – Shake Table Fidelity</td>
<td>413</td>
</tr>
<tr>
<td>I</td>
<td>Selected Seismic Results – Visual Damage</td>
<td>420</td>
</tr>
<tr>
<td>J</td>
<td>Selected Seismic Results – Displacement and Acceleration</td>
<td>427</td>
</tr>
<tr>
<td>K</td>
<td>Selected Seismic Results – Base Shear Force-Displacement Hysteresis Loops</td>
<td>464</td>
</tr>
<tr>
<td>L</td>
<td>Selected Seismic Results – Peak Anchor Bolt Forces</td>
<td>473</td>
</tr>
<tr>
<td>M</td>
<td>Selected Seismic Results – Peak Sill Plate and Holdown Stud Uplift</td>
<td>510</td>
</tr>
<tr>
<td>N</td>
<td>Selected Seismic Results – Sill Plate Slippage</td>
<td>547</td>
</tr>
<tr>
<td>O</td>
<td>Comparison of Experimental and Numerical Results for Phase 9</td>
<td>584</td>
</tr>
<tr>
<td>Chapter 14</td>
<td>References</td>
<td>597</td>
</tr>
</tbody>
</table>
SCOPE OF RESEARCH

A two-story single-family woodframe house was tested using the UC San Diego uniaxial earthquake simulation system under Task 1.1.1 of the CUREe-Caltech Woodframe Project. The main objectives of the study were to determine the dynamic characteristics and the seismic performance of the test structure under various levels of seismic shaking and structural configurations. The structure was tested during 10 phases of construction to determine the performance of the structure with fully sheathed shearwalls, symmetrical and unsymmetrical door and window openings, perforated shearwall construction, conventional construction, and with and without non-structural wall finish materials. Four types of shake table tests were performed: quasi-static in-plane floor diaphragm tests, frequency evaluation tests, damping evaluation tests, and seismic tests.

The test results showed excellent performance of the fully engineered test structure. However, the test structures using perforated shearwall construction and conventional construction showed increased dynamic response as compared to the engineered structure. Window and door openings caused a reduction in lateral stiffness of the structure and a corresponding increase in seismic response as compared to the fully sheathed structure. Torsional behavior was evident in the structure with non-symmetrical openings. Non-structural wall finish materials caused a large increase in lateral stiffness of the structure with a corresponding decrease in seismic response. A numerical model, based on non-linear time-history dynamic analyses, was also developed to predict the seismic response of the structure.
REPORT LAYOUT

This report is organized into chapters discussing the test structure and results as well as appendices providing other relevant information about the test structure and selected results. After introducing the project in Chapter 1, Chapter 2 provides a literature review of previous full-scale testing of woodframe structures. Chapter 3 gives a description of the test structure and its structural components as well as the testing objectives. Chapter 4 discusses the shake table tests performed for each phase of testing. Chapter 5 provides descriptions of the instrumentation used during the shake table tests. Chapter 6 includes descriptions of material property testing including moisture content testing of wood materials, sheathing-to-framing connection testing, and compressive testing of exterior stucco. The results of the quasi-static tests are presented in Chapter 7. Chapter 8 discusses the frequency evaluation test results. The damping evaluation test results are provided in Chapter 9. The results of the seismic tests are discussed in Chapter 10. A numerical model is developed and compared with experimental results in Chapter 11. Conclusions regarding the shake table testing and numerical modeling of the structure are presented in Chapter 12. Chapter 13 includes appendices for information not included in the main body of the report. Each of the appendices is discussed in the respective chapters. Chapter 14 presents a list of references.
Chapter 1 Introduction

As a result of the 1994 Northridge Earthquake, there was $20 billion in property loss to woodframe construction. The damage and loss to woodframe construction was considerable higher than other types of construction (reinforced concrete, steel, or masonry). The majority of insurance claims paid out following the Northridge Earthquake were for residential claims. More than 80% of the building structures in Los Angeles County were woodframe construction at the time of the earthquake according to County Assessor data. In addition, approximately 99% of the residences in Los Angeles County were woodframe Construction. In the Northridge Earthquake, hillside houses and multi-story “apartments-over-garage” multi-family buildings suffered more damage compared to average single-family compared
to average single-family houses. These structures varied in age, but the majority of the woodframe structures were constructed in recent decades in which building codes in California required seismic provisions.

Despite the seismic provisions in building codes in California, many structural engineers and researchers believe that the design methodologies required by current codes are based on individual component behavior (shearwalls or diaphragms) and do not capture the true behavior of a woodframe structure at the system level during a seismic event. In addition, the reliability in the performance of woodframe structures is significantly lower than other types of construction. In concrete and steel structures, there has been extensive experimental testing and codifications of a design philosophy such that structural damage can be controlled. This design philosophy does not exist in woodframe construction. As a result, the CUREe-Caltech Woodframe Project was developed to address these shortcomings of woodframe construction.

1.1 Description of the CUREe-Caltech Woodframe Project

The main objective of the CUREe-Caltech Woodframe Project is to create reliable and economical ways of improving the seismic performance of woodframe construction. The project is funded by the Federal Emergency Management Agency (FEMA) through the California Office of Emergency Services (OES). The primary contractor of the project is the California Institute of Technology (Caltech). California Universities for Research in Earthquake Engineering (CUREe) has coordinated and subcontracted the non-Caltech work of the project. A flowchart showing the project management of the CUREe-Caltech Woodframe Project is shown in Figure 1.1.

The project has been divided into five separate elements. Element 1, Testing
and Analysis, involves experimental testing of large-scale woodframe structures and structural components. Element 2, Field Investigations, is concerned with taking reported data and observations from recent earthquakes and converting it to a scientific form that can be used to help improve current building codes. Element 3, Building Codes and Standards, involves using the experimental data and field investigations to improve the current building codes. Element 4, Economic Applications, is concerned with using information from this project to refine insurance, mortgage lending, loss estimation, and disaster relief polices and procedure for woodframe construction. Element 5, Education and Outreach, involves educating building owners and residents to initiate actions to improve existing woodframe

Figure 1.1: Project Management of the CUREe-Caltech Woodframe Project
structures that may be hazardous during an earthquake.

The project includes also an Advisory Committee that is comprised of representatives from various professions including the construction industry, insurance-finance, building code representatives, engineers, and state and local agencies from within California and nationally as well. The main objective of the committee is to provide support to the project managers on the current design practice and other trends in woodframe construction.

Since this report is part of Element 1 of the CUREe-Caltech Woodframe Project, this element will be discussed in further detail. As part of Element 1, three full-scale shake table tests were conducted in an effort to determine the performance of full-scale woodframe structures under seismic events at the system level. Under Task 1.1.1, a two-story single-family house was tested under seismic events using a shake table. Task 1.1.1 is the project presented in this report. For Task 1.1.2, a three-story apartment building with parking on the first story was tested using a shake table. Under Task 1.1.3, a simplified box-shaped woodframe structure was tested using a shake table. Figure 1.2 shows a flowchart of the organization of the various tasks of Element 1.

Supporting the full-scale shake table tests, there are several component test tasks including shearwalls, anchorage, diaphragms, cripple walls, wall finish materials, and rate of loading and loading protocol effects. Of special importance to this project (Task 1.1.1) is Task 1.3.2 Testing Protocols and Task 1.5.1 Analysis Software. Under Task 1.3.2, the ground motions for the seismic tests of test structure
1.1.1 Single-Family House (UC-San Diego)  
1.1.2 Apartment Building (UC-Berkeley)  
1.1.3 Simplified Model (British Columbia)  
1.2 International Benchmark (UC-San Diego)  
1.3.1 Rate of Loading & Loading Protocol Effects (UC-San Diego)  
1.3.2 Testing Protocols (Stanford)  
1.3.3 Dynamic Characteristics (Caltech)  
1.4.1 Anchorage (WJE, USC)  
1.4.2 Diaphragms (Virginia Tech)  
1.4.3 Cripple Walls (UC-Davis)  
1.4.4 Shear Walls (UC-Irvine)  
1.4.6 Wall Finish Materials (Stanford) (San Jose State)  
1.4.7 Innovative Systems (Washington State)  
1.4.8 Connections (Brigham Young) (UC-Irvine) (Washington State)  
1.5.1 Analysis Software (UC-San Diego)  
1.5.2 Demand Aspects (Stanford)  
1.5.3 Reliability Analysis (Oregon State)  
1.5.4 Analysis of Index Buildings (UC-San Diego)  

Figure 1.2: Flowchart of Element 1 of the CUREe-Caltech Woodframe Project

and cyclic loading protocol for connection tests were developed. Under Task 1.5.1, a software package was developed to predict the hysteretic properties of a shearwall that was used in the numerical modeling of the two-story structure. Task 1.1.1 will be discussed in further detail in the following section.

1.2 Description of Task 1.1.1: Shake Table Tests of a Simplified Two-Story Single Family House

As discussed previously, shake table tests were performed on a simplified full-scale two-story single-family house under Task 1.1.1 of the CUREe-Caltech Woodframe Project. The main objective of this testing was to quantify the dynamic response of the structure with various structural configurations. Since there has been a lack of understanding of the behavior of full-scale structural woodframe systems,
these shake table tests attempt to improve this understanding. In addition, this project generated extensive experimental data at the system level.

The test structure is 16 ft by 20 ft in footprint and is anchored to the UC San Diego uniaxial shake table such that shaking occurs along the short dimension of the structure (North-South direction). The structural components of the test structure are full-scale, however the plans dimensions of the test structure are smaller than a typical residence due to restrictions of the shake table. The lateral load resisting system consists of exterior shearwalls sheathed with Oriented Strand Board (OSB). The structure was tested during 10 different phases of construction with varying diaphragm properties and shearwall configurations. For each of the first four phases of testing, quasi-static tests on the structure were performed to determine the in-plane stiffness of the floor diaphragm. For the last six phases of testing, quasi-static tests, frequency evaluation tests, damping evaluation tests, and seismic tests were performed on the test structure. The test structure was repaired between each phase of testing to return the lateral load resisting system to its initial strength and stiffness. The test structure was instrumented with nearly 300 displacement, acceleration, and force measuring devices to maximize the amount of information regarding the response of the structure during the various tests.
Chapter 2  Literature Review on Full-Scale Testing of Woodframe Structures

2.1  Review of Previous Studies

Until recently, the majority of woodframe houses in North America were built based on experience and tradition. Significant damage to residential woodframe construction from earthquakes and high winds has created a need to examine the current design practices and building codes for woodframe construction. During the last 50 years, there has been significant testing on individual woodframe building components such as shearwalls and diaphragms. However, there has been little testing on full-scale woodframe construction. It is important to quantify the interaction of the
individual building components during lateral loading of full-scale woodframe construction. The full-scale testing of woodframe houses that has been performed to date can be divided into lateral wind load testing and shake table testing. Table 2.1 summarizes the experimental research conducted to date on full-scale testing of woodframe construction.

Beginning in the 1950’s, researchers tested woodframe houses subjected to simulated wind loads. Houses tested in the United States were constructed using typical two-by-four wall construction. Houses tested in Japan were constructed using post and beam construction. The interior surfaces of the walls and ceilings were covered with gypsum wallboard or sheetrock. The exterior surfaces of the walls were sheathed with wood or metal siding. Diagonal braces at the corners of the building were used to resist lateral loads in the structures. In earlier testing, the roofs of these houses were sheathed with evenly spaced boards. In more recent testing, the roofs were sheathed with plywood. The houses were either loaded with several hydraulic rams producing concentrated lateral loads, several hydraulic rams in a whiffle tree arrangement producing distributed lateral loads, or with an air bag producing distributed lateral loads. The majority of the full-scale woodframe structures tested with simulated wind loads preformed well without catastrophic failure. The horizontal floor and roof diaphragms had small in-plane deformations suggesting that they behaved more like rigid diaphragms. Non-structural finish materials greatly increased the lateral stiffness of the structures. The diagonal braces did not resist a significant portion of the lateral load when gypsum wallboard and exterior wall siding were installed.
### Table 2.1: Summary of Experimental Research on Full-Scale Woodframe Structures

<table>
<thead>
<tr>
<th>Reference</th>
<th>Test Specimen</th>
<th>Loading</th>
<th>Focus of Research</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dorey and Schriever (1957)</td>
<td>Single-story stud-framed house</td>
<td>Static simulated wind and snow loads</td>
<td>Investigate structure response without exterior sheathing</td>
</tr>
<tr>
<td>Hurst (1965)</td>
<td>Single-story stud-framed house</td>
<td>Quasi-static cyclic simulated wind loads</td>
<td>Investigate contribution of roof, walls, partitions, and floor to structure</td>
</tr>
<tr>
<td>Yokel et al. (1973)</td>
<td>Two-story stud-framed house</td>
<td>Static and dynamic simulated wind loads</td>
<td>Investigate story drift and dynamic response characteristics of structure</td>
</tr>
<tr>
<td>Tuomi and McCutcheon (1973)</td>
<td>Single-story stud-framed house</td>
<td>Quasi-static cyclic concentrated lateral loads</td>
<td>Investigate racking resistance of structure at several stages during construction</td>
</tr>
<tr>
<td>Stewart et al. (1988)</td>
<td>Single story manufactured home</td>
<td>Static concentrated and uniform lateral loads</td>
<td>Investigate racking resistance of transverse walls and interaction of roof</td>
</tr>
<tr>
<td>Sugiyama et al. (1988)</td>
<td>Two-story Japanese-style house</td>
<td>Quasi-static cyclic concentrated lateral loads</td>
<td>Investigate influence of commonly ignored structural factors on structure</td>
</tr>
<tr>
<td>Yasamura et al. (1988)</td>
<td>Three-story Japanese style house</td>
<td>Quasi-static monotonic and cyclic concentrated lateral loads</td>
<td>Investigate structural safety of three-story woodframe house subjected to lateral loads</td>
</tr>
<tr>
<td>Carydis and Vougioukas (1989)</td>
<td>Two-story Greek-style timber frame house</td>
<td>Earthquake ground motion using a shake table</td>
<td>Demonstrate that wood houses are adequately resistant to earthquakes</td>
</tr>
<tr>
<td>Phillips et al. (1993)</td>
<td>Single-story stud-framed house</td>
<td>Quasi-static cyclic concentrated lateral loads</td>
<td>Investigate how horizontal diaphragm distributes lateral load into shearwalls</td>
</tr>
<tr>
<td>Tanaka et al. (1998)</td>
<td>Two-story post and beam house</td>
<td>Earthquake ground motion using a shake table</td>
<td>Investigate structural safety and seismic performance of post and beam houses</td>
</tr>
<tr>
<td>Kohara and Miyazawa (1998)</td>
<td>Two-story Japanese-style house</td>
<td>Earthquake ground motion using a shake table</td>
<td>Investigate damage resulting from earthquake ground motions and quantify dynamic behavior of structure</td>
</tr>
<tr>
<td>Ohashi et al. (1998)</td>
<td>Single-story Japanese style house</td>
<td>Earthquake ground motion using a shake table</td>
<td>Investigate structural safety during an earthquake and quantify the influence of nonstructural finish materials on structure stiffness</td>
</tr>
<tr>
<td>Yamaguchi and Minowa (1998)</td>
<td>3.64 m (12 ft) long bearing walls</td>
<td>Earthquake ground motion using a shake table</td>
<td>Compare hysteretic behavior of bearing walls from shake table testing with the hysteretic behavior of bearing walls from static testing</td>
</tr>
<tr>
<td>Seo et al. (1999)</td>
<td>Single-story one-quarter scale post and beam house</td>
<td>Earthquake ground motion using a shake table</td>
<td>Determine the maximum peak ground acceleration an ancient Korean commoner's house could sustain without collapse</td>
</tr>
<tr>
<td>Takahiro et al. (2000)</td>
<td>Single-story Japanese style house</td>
<td>Earthquake ground motion using a shake table</td>
<td>Determine the influence of eccentricity and in-plane diaphragm stiffness on seismic performance</td>
</tr>
</tbody>
</table>

In order to provide a better perspective on past experimental research on full-scale wood structures, the key results emerging from the references provided in
Table 2.1 are briefly discussed below. Beginning in the late 1980’s, researchers began testing full-scale woodframe houses subjected to earthquake ground motions on shake tables. All of these shake table tests were preformed outside of the United States: in Japan, Korea, and Greece. These houses were typically post and beam construction with diagonal braces designed to resist the lateral inertia loads. The exterior wall surfaces were covered with wood or fiberboard siding or mortar stucco. The interior wall and ceiling surfaces were typically covered with gypsum wallboard. The floors and roofs of these structures were sheathed with plywood. The houses were tested with several structural configurations during the construction process. The houses were subjected to several ground motions from recent earthquakes and variable frequency motions to quantify the dynamic response of the structures. These structures performed well when subjected to the earthquake ground motions with only one catastrophic failure. Again, the non-structural finish materials greatly increased the lateral stiffness of the structures. Without gypsum wallboard and exterior siding, the diagonal braces were not capable of resisting the lateral loads in the structures. Researchers were able to predict the experimental response of the structures using a bi-linear hysteresis for shearwalls in analytical models.

**Study by Dorey and Schriever (1957)**

In 1954, Dorey and Schriever (1957) of the Division of Building Research of Canada tested a full-scale single-story house subjected to simulated wind and snow loads. The main objective was to quantify the strength and rigidity of a full-scale house without exterior sheathing. Another objective was to use the resulting
experimental data in writing performance requirements for houses in Canada. Previous researchers had tested individual building components, but this testing did not investigate the interaction of the building components in a three-dimensional structure. The house was built in the field in 1948 for experimental purposes, but not specifically for structural testing. The single-story house was 24 ft – 6 in (7.5 m) wide (direction of loading) by 36 ft – 8 in (11.2 m) long. The woodframe structure was built on a basementless block wall foundation that extended 3 ft to 4 ft (0.9 m to 1.2 m) below grade. Two-by-six sill plates were attached to the top of the foundation wall using anchor bolts. The floor system was constructed using two-by-eight joists at 16 in (406 mm) on center supported at mid-span by a built-up timber beam. The floor was sheathed with 13/16 in (21 mm) thick diagonal boards with ¾ in (19 mm) thick hardwood flooring. The exterior walls were constructed using two-by-four studs at 16 in (406 mm) on center with one-by-four let-in bracing at the corners. These walls were covered with 12-pound asphalt-saturated felt paper and interlocking aluminum siding on the exterior. The interior side of these walls was covered with 3/8 in (10 mm) thick gypsum wallboard. Most interior partition walls were constructed using two-by-four studs at 16 in (406 mm) on center with 3/8 in (10 mm) thick gypsum wallboard on both sides. As an experiment, a few interior walls were constructed using only 2 in (51 mm) thick solid plaster. The roof was framed using two-by-four rafters at 16 in (406 mm) on center with one-by-four collar ties. The roof framing was sheathed with one-by-six boards spaced 6 in (152 mm) apart with aluminum roofing. The ceiling was constructed with two-by-sixes supported at mid-span by an interior partition wall and was sheathed with 3/8 in (10 mm) thick gypsum wallboard.
A rigid reaction frame was constructed using Bailey bridging to span across the structure. Hydraulic rams were used to apply loads to the roof and walls using a “whiffletree” arrangement. Hardwood loading pads were used at each point of loading to further distribute the load. The house was tested four times with increasing severity of load during each test. During the first test, the structure was loaded to simulate wind with internal suction representing a condition with windows open on the leeward side only. During the second test, the structure was loaded to simulate wind with internal pressure representing a condition with windows open on the windward side only. For the third test, researchers tested the structure with simulated wind loads and one-half of the design snow load. For these tests, the structure was loaded with simulated winds starting at 70 mph (113 km/h) and ending at 120 mph (193 km/hr) increasing in 10 mph (16 km/hr) increments. The load was maintained for one-half hour for each of the first three load increments and one hour for each of the final three load increments for each test. During the fourth test, simulated snow loads were applied to the roof in increments of 13 psf (0.6 kPa) (25% of the design snow load) to a maximum of 73 psf (3.5 kPa) (143% of the design snow load). Again, the load was maintained for one-half hour at each load increment.

Deflections of the structure were measured using a system of pulleys and wires. Deflection measurements were taken at several locations on the transverse end walls (direction of loading) and intermediate transverse vertical planes. At each measurement location, a piano wire was run over low-friction aircraft pulleys out a window to a instrument hut. Each of the 43 wires was tensioned with a one-pound weight strung across a deflection board. In addition to the deflection measurements,
plaster telltales were installed at the ceiling level of several interior corners in the structure to observe damaging deformations. The plaster telltales were made by replacing the paint from the intersecting planes at a corner with several layers of gypsum plaster.

The structure performed well when subjected to wind loading. In the first test of simulated winds with internal suction, no damage resulted at equivalent wind velocities of 90 mph (145 km/hr). Cracking became apparent in the plaster telltales as the simulated winds were increased to 120 mph (193 km/hr). During this first test, the maximum lateral displacement of the structure was 0.12 in (3 mm) and the largest bending deflection in the roof rafters was 0.31 in (8 mm). In the second test of winds with internal pressure, a few of the existing cracks in the plaster telltales increased in width. The maximum lateral displacement of the structure was 0.08 in (2 mm) and the maximum bending deflection in the rafters was 0.80 in (20 mm). No further significant damage resulted from the third test with simulated wind and one-half of the design snow load. Since the snow load acted in the opposite direction of the simulated wind load on the roof, the largest deflection in the rafters occurred at the minimum wind load for this test. In the final test with simulated snow loads, a few collar ties failed at a load of approximately 73 psf (3.5 kPa) (143 percent of the design snow load). The allowable deflection of L/240 was exceeded on the front roof slope by 182 percent during this test. The wind tests performed on the structure showed that the structure was capable of withstanding 1.8 times the design wind load using only one-by-four let-in bracing and interior finish materials to resist lateral loads.
**Study by Hurst (1965)**

Between 1963 and 1964, Hurst (1965) conducted a study of a full-scale woodframe house in cooperation with the National Forest Products Association. Deflection of floors subjected to simulated gravity loads and the racking resistance of walls subjected to simulated wind loads were studied. However, the discussion here will be limited to the experimental procedure and results developed from the lateral load testing of the structure. The main objective of the study was to determine the structural performance of a conventional full-scale, woodframe house under simulated wind loads acting diagonally on the structure. At the time of the experiment, residential wood structures were designed on a piece-by-piece basis without consideration of three-dimensional effects. The researchers wanted to investigate the structural contribution of various building components such as the roof, walls, partitions, and floors to the overall performance of the complete structure.

The test structure was 36 ft (11.0 m) long by 28 ft (8.5 m) wide with three bedrooms, a living and dining room area, kitchen, and one and a half bathrooms. The house was built in a laboratory on concrete block walls that formed a basement. Two-by-eight floor joists spliced over a middle laminated girder that spanned the transverse direction of the house. The floor was sheathed with one-by-eight diagonal sheathing and the basement ceiling was covered with ½ in (13 mm) thick sheetrock. The walls were constructed using typical two-by-four construction and were sheathed with 3/8 in (10 mm) thick plywood on the exterior and ½ in (13 mm) thick sheetrock on the interior. Interior partition walls were framed using two-by-fours with ½ in (13 mm)
thick sheetrock on both sides. The roof was framed with manufactured roof trusses spaced at 24 in (610 mm) and was sheathed with one-by-eight boards.

The wall and diaphragm systems were tested with simulated wind loads during 7 stages of dismantling of the structure. The completed structure sheathed with plywood on the exterior and sheetrock on the interior was tested during Stage 1. Prior to Stage 2 testing, the sheetrock joints were taped and plastered. The sheetrock was removed from all partition walls for Stage 3. The partition walls were removed for Stage 4. For Stage 5 testing, the sheetrock was removed from the exterior walls. The one-by-eight roof sheathing was removed prior to testing in Stage 6. For the last stage, Stage 7, the gypsum wallboard was removed from the roof ceiling except for a 4-ft (1.2 m) wide strip adjacent to the two leeward walls.

The structure was loaded cyclically with a simulated wind load acting diagonally on the house. Hydraulic load cylinders were used to simulate uniform wind pressure on the walls with concentrated loads at 16 in (406 mm) on center and on the roof with concentrated loads at 24 in (610 mm) on center. All roof and wall surfaces were subjected to the same simulated wind pressures up to a maximum of 16 psf (766 Pa) loaded in 4 psf (192 Pa) increments. For each load increment, the load was cycled on and off three times during six-minute cycles. For each load increment, a gravity load was imposed on the roof equal to the simulated wind pressure. Horizontal deflection recorders measured deflection at 19 points on the two leeward walls.

The structure performed well up to simulated wind loads of 16 psf (766 Pa). The researchers initially intended to test the structure to simulated wind loads much greater than 20 psf (958 Pa), but the foundation wall failed at a load of 20 psf (958 Pa).
Pa). Maximum wall deflections occurred midway between the floor and top plates during all stages of testing. The researchers found that the taping and plastering of the walls prior to Stage 2 had little effect on the horizontal deflection of the structure. In addition, there was little change in the horizontal deflections as a result of removing the sheetrock from the interior partitions for Stage 3. Removing these partitions before Stage 4 had little effect on the horizontal deflections as well. After the sheetrock was removed from the exterior walls for Stage 5, again, there was little change in the horizontal movement of the building. However, when the roof sheathing was removed for Stage 6, there was a large increase in horizontal deflection of the structure. During Stage 7 with the sheetrock removed from the majority of the ceiling, a simulated wind load of 12 psf (575 Pa) caused the end wall to pull away from the 4-ft (1.2 m) wide strip of sheetrock. From these tests, the researchers concluded that the ceiling and the roof sheathing were very important structural elements in limiting horizontal movement. However, since the 3/8 in (10 mm) plywood sheathing was much stiffer than the interior sheetrock, researchers urged that conclusions should not be made about the effect of the taping and plastering of the sheetrock and the interior partitions on the racking resistance of the structure.

**Study by Yokel et al. (1973)**

In 1972, Yokel et al. (1973) tested a full-scale two-story house that was representative of housing in the United States. A large producer of conventional housing donated a recently completed house located in Maryland for testing. In this investigation, the main objective was to determine whether existing drift limitations
for medium-rise and high-rise structures were applicable to conventional low-rise housing. Another objective was to quantify the dynamic response characteristics of conventional housing to improve dynamic lateral load calculations in design.

The two-story woodframe test structure was 47 ft (14.3 m) long by 26 ft (7.9m) wide (direction of loading). The house contained a family room, bedroom, bathroom, and a garage on the lower story and a L-shaped living and dining room, kitchen, three bedrooms, and two bathrooms on the upper story. The exterior walls were constructed with two-by-four studs and the interior walls were constructed using a combination of two-by-four and two-by-three studs. All interior wall surfaces were covered with 3/8 in (10 mm) thick gypsum wallboard. The exterior wall surfaces were covered with ½” (13 mm) thick gypsum sheathing with asbestos shingles and 3/8 in (10 mm) thick beveled wood siding. The exterior walls were braced at the corners with one-by-four let-in bracing installed at a 45-degree angle. The second floor was framed with two-by-eights spaced at 12 in (305 mm) with 5/8 in (16 mm) thick plywood sub-flooring and ½ in (13 mm) thick gypsum wallboard on the ceiling below. The roof was framed with two-by-four trussed roof rafters at 24 in (610 mm) on center and was sheathed with ½ in (13 mm) thick plywood and asphalt shingles.

The test structure was loaded over a series of four static tests and one dynamic test. The static tests were intended to measure the stiffness of the house when subjected to simulated wind loads in the transverse direction. Four single-acting hydraulic rams with a load capacity of 10 tons were used in the static tests. During three of the tests, the concentrated loads were applied horizontally at the centerline of the second story floor joists. The concentrated loads were applied at the bottom chord
of the roof trusses during the other static test. In each of these tests, the load was
cycled producing maximum total loads between 2 kips (8.9 kN) and 10 kips (44.5 kN).
The dynamic test was intended to measure the dynamic response characteristics of the
structure. During this test, a 12” long piece of steel pipe was installed between the
hydraulic ram and the house. After each ram was loaded, the steel pipe was removed
with a sharp blow of a hammer and the structure was allowed to vibrate freely. This
test had to be repeated several times because of limited recording equipment.
Thirty-two displacement transducers were used to measure the overall building
movement and the racking deformation of the walls. Two of the transducers were
more sensitive for use in the dynamic response test of the house. Since the structure
was tested during a time of inclement weather, all of the recording devices and
equipment had to be installed on the interior of the structure.

During the static tests, it was found that the walls behaved nearly elastically.
Thus, the results of the upper story testing and the lower story testing could be
superimposed on one another. A wind load of approximately 15 psf (718 Pa) was
simulated using a total ram load of 2.82 kips (12.5 kN) at the bottom chord of the roof
truss and a total ram load of 5.64 kips (25.1 kN) at the second story floor joists. The
simulated wind load on the structure was calculated in accordance with ANSI
Standard A58.1-1955. As a result of these tests, it was believed that the drift at the
upper story was much less than that calculated using design criteria for medium-rise
and high-rise structures. The let-in braces used at the corners resisted the majority of
the racking loads because only a small portion of the exterior wall distortion was
transmitted to the interior gypsum wallboard. The roof diaphragm underwent
significant in-plane deformations suggesting it behaved more like a flexible diaphragm. However, the second floor diaphragm acted more like a rigid body during loading of the structure. During the dynamic tests, the natural frequency of the house was around 9 Hz. The percentage of critical damping ranged from four to nine percent of critical damping with an average of six percent. However, the researchers urged that the conclusions regarding the dynamic properties should be questioned because of the resolution of the dynamic recording equipment.

**Study by Tuomi and McCutcheon (1973)**

Tuomi and McCutcheon (1973) tested a full-scale single story house under simulated snow and wind loads in a laboratory. The main objective was to determine the response of a typical woodframe house subjected to horizontal wall loads and vertical roof loads. The main objective was divided into four specific goals. The first goal was to quantify the racking resistance of shearwalls subjected to horizontal concentrated loads during several stages of construction. The second goal was to investigate the serviceability limits in which door and windows became inoperable and cracks in the wall coverings became prevalent. The third goal was to correlate the racking resistance of a structure subjected to concentrated horizontal loads at the shearwalls with distributed horizontal loads on a wall perpendicular to the direction of loading. The final goal was to determine the ultimate cause of structural failure.

The test structure was 24 ft (7.3 m) long (direction of loading) by 16 ft (4.9 m) wide. The house was constructed with typical materials used in 1973 meeting the minimum size requirement according to the Federal Housing Administration.
Minimum Property Standards, FHA No. 300. The 24-ft (7.3 m) length of the house was typical of residential construction. However, the 16-ft (4.9 m) width of the house was only about one-half of the width of a typical house. The two 24-ft (7.3 m) long gable-end walls (direction of loading) each contained a window and a door. In the west gable-end wall, the door opening was located near the north end of the wall whereas the door opening in the east gable-end wall was located near the south end of the wall. The two north and south end walls did not contain openings. Four wide flange beams supported the four-wall box-shaped test structure. Two-by-six sill plates were bolted to the wide-flange beams with ½ in (13 mm) diameter bolts at 8-ft (2.4 m) intervals. The walls were constructed using two-by-fours (stud grade Douglas-fir) at 16 in (406 mm) on center with double top plates and intermediate wall blocking. The walls were sheathed with 3/8 in (10 mm) plywood with 6-penny nails spaced at 6 in (152 mm) around the perimeter of the panels and at 12 in (305 mm) to intermediate supports. The interior side of each wall was covered with ½ in (13 mm) gypsum wallboard with 1-1/4 in (32 mm) ring-shank nails at 8 in (203 mm) on center. Western red cedar siding was installed horizontally to the exterior of the structure. The roof was constructed with manufactured roof trusses spaced at 24 in (610 mm). The roof was sheathed with 3/8 in (10 mm) plywood with 6-penny nails spaced at 6 in (152 mm) around the perimeter of the panels and at 12 in (305 mm) to intermediate supports. The ceiling was covered with ½ in (13 mm) gypsum wallboard using 1-1/4 (32 mm) ring-shank nails at 6 in (152 mm) on center.

The house structure was tested with lateral loads at six stages during construction. In the first five tests, shearwall deflections were limited in an effort to
keep the walls in their elastic range. Concentrated loads were applied to the top of the longitudinal shearwalls for the first five tests. During Stage 1, the initial stiffness of the longitudinal walls was measured with only 3/8 in (10 mm) plywood sheathing on the exterior. Windows and doors were installed in Stage 2 to evaluate the reduction of stiffness with openings. The interior gypsum wallboard and cedar siding were added for the testing in Stage 3. During Stage 4, the transverse walls were constructed to evaluate the influence of the walls perpendicular to the direction of loading. The roof system was constructed for Stage 5 to quantify the racking resistance of a complete three-dimensional structure. For Stage 6, a uniform load was applied to the south transverse wall to determine the stiffness, ultimate strength, and load distribution for a structure subjected to wind loading. Since there was little structural failure from to the six lateral tests, vertical load tests on the roof were also conducted. The roof was loaded with a “whiffletree” arrangement using four jacks to produce 32 equal concentrated loads.

During the concentrated lateral load testing, loads were applied in 0.025-in (0.6 mm) displacement increments with hydraulic jack actuators. The load was applied at a rate greater than or equal to two minutes per increment. After each new level of deformation, the load was reduced to zero and residual displacements were measured after two minutes. The maximum lateral displacements were kept between 0.125 in (3 mm) and 0.150 in (4 mm) for the first five test stages. For the uniform load testing of Stage 6, an air bag system was used to apply lateral loads to the south transverse wall. The transverse wall was loaded in increments of 5 psf (0.24 kPa) up to a maximum pressure of 40 psf (1.9 kPa). Once this maximum pressure of 40 psf (1.9 kPa) had been
reached, the load was cycled three times to this level. Following the third cycle, the lateral uniform load was increased until failure. For the vertical load roof testing, the roof was loaded to 16,000 lbs (71.2 kN) (approximately 45 psf) in 4,000-lb (17.8 kN) increments and then cycled four times at this level. The load was then increased again in 2,000-lb (8.9 kN) increments until failure.

It was found that the racking resistance of the structure was more than adequate. The load-displacement relationships were found to be curvilinear as opposed to the linear elastic relationship that was assumed. After the first two cycles of load, there was residual set in the walls due to the inelastic nail-slip. As a result of the ultimate uniform load testing of Stage 6, component failure first occurred when sole plate on the loaded transverse wall broke loose at a lateral uniform load of 63 psf (3.0 kPa). In addition, component failure occurred at a sill plate joint at lateral uniform load of 123 psf (5.9 kPa). At a load of approximately 70 psf (3.4 kPa) (corresponding to a wind speed of 165 mph), a center stud on the loaded transverse wall failed in bending. After comparing the concentrated load testing to uniform load testing, it was discovered that three-fourths of the lateral load was resisted by the shearwalls and the other one-fourth of the lateral load went directly into the foundation. In terms of serviceability, one of the doors began to bind at a lateral deflection of 0.10 in (2.5 mm). At one inch of bending deflection in the loaded transverse wall, the gypsum wallboard showed no significant damage. However, the gypsum wallboard joints were not taped. The trusses performed very well when subjected to simulated snow loads with a failure load of 135 psf (6.5 kPa).
Study by Stewart et al. (1988)

Stewart et al. (1988) tested two manufactured homes under simulated wind loads. The main objective of this testing was to evaluate the lateral load capacity of a manufactured home. Specifically, the researchers wanted to determine how the transverse walls (direction of loading) provided racking resistance under lateral loads. They also wanted to determine the interaction between the roof diaphragm and the shearwalls. Lastly, the researchers wanted to evaluate the loading protocol for use in future testing of manufactured homes.

The two identical 13.7 ft (4.2 m) wide by 66 ft (20.1 m) long homes were constructed under standard production line conditions. However, non-structural components such as cabinets, plumbing, and electrical components were not installed for ease in instrumentation of the structure. The transverse wall system was designed to resist a lateral load of 25 psf (1.2 kPa) through five shearwalls. The roof of each manufactured home was framed using trusses and covered with metal roofing and a fiberboard ceiling.

A total of 78 electronic instruments were used to record the behavior of each structure under lateral loads. Electronic transducers were used to measure horizontal and vertical deflection of the shearwalls and roof diaphragm. Load cells and pressure sensors were used to measure applied and internal loads. Strain gages were used to monitor loads in connector straps and lag bolts that connected the floor of the homes to the chassis below. Lastly, eight dial gages and 2 pendulums were used to monitor box rotation.
The structures were loaded in two different sequences. The first sequence of loading involved applying concentrated loads with hand-operated hydraulic actuators to each of the five transverse walls individually. The loads were applied slowly to allow researchers to record loads and deformations at each increment. Loads up to a maximum of 3,500 lbs (15.6 kN) were applied to the top and bottom of each of the shearwalls. This concentrated load testing was used to estimate the in-place stiffness of the shearwalls. In the second sequence of lateral loading, uniform lateral loads were applied to the longitudinal side of the building to simulate wind loading. Air bags pushing against a reaction wall and the structure provided the distributed load up to 75 psf (3.6 Kpa). The pressure in the air bags was measured using a water manometer and a pressure sensor. In addition, eight load cells were used to verify the pressure measurements.

The concentrated load tests indicated that the roof diaphragm had a high stiffness compared to the five shearwalls. Since the diaphragm acted rigidly, the majority of the lateral load was transferred to the stiffer end walls. The uniform load tests revealed that both test structures had a much higher ultimate capacity than the design value of 25 psf (1.2 kPa). The first test house had an ultimate lateral capacity of 75 psf (3.6 kPa) producing a safety factor of 3.0. Failure occurred in one of the end shearwalls, but the structure continued to resist load since the failure was not catastrophic. The second test structure had an ultimate lateral capacity of 71 psf (3.4 kPa). Testing was stopped at this point due to the failure of a tie-down strap. During these tests, it was found that the shearwalls exhibited a nearly linear, elastic response. It was concluded that the current design practice considering the interaction of the
shearwalls, roof diaphragm, floor diaphragm, and chassis in the design of manufactured homes was satisfactory. It was also concluded that the testing protocol worked well and should be used in future testing.

**Study by Sugiyama et al. (1988)**

Sugiyama et al. (1988) tested a full-scale Japanese style two-story house subjected to lateral loads. The main objective of this experimental investigation was to evaluate the influence of commonly ignored structural factors on the racking resistance of a structure. The researchers wanted to quantify the influence of wall sheathing above and below door and window openings on the racking resistance of the wall. In addition, they examined the effect of shear frames placed perpendicular to the direction of lateral loading.

The test structure was a full-size two-story house 7.28 m (24 ft) wide (direction of loading) by 10.01 m (33 ft) long. Four shearwall frames resisted lateral loads in the direction of loading. The frames were constructed using 12 cm by 12 cm (4 ¾ in by 4 ¾ in) wood columns connected with 12 cm by 33 cm (4 ¾ in by 13 in) beams at the second floor. The frames were connected with 12 cm by 30 cm (4 ¾ in by 12 in) sub-beams spaced at 1.82 m (6 ft). The second floor was framed with 4.5 cm by 9.0 cm (1 ¾ in by 3 ½ in) joists and spaced at 22.5 cm (9 in) and sheathed with 3-ft by 6-ft (91 cm by 183 cm) sheets of 1.2 cm (1/2 in) thick lauan plywood. A large opening was left in the second floor for a staircase. Studs with dimensions of 12 cm by 3 cm (1 ¼ in by 4 ¾ in) were installed between posts at 45.5 cm (18 in) on center. Let-in bracing in each of the shearwall frames resisted lateral loads. The walls were sheathed with
exterior calcium silicate wall siding during certain stages of testing. The hip roof was sheathed with 3-ft by 6-ft (91 cm by 183 cm) sheets of 1.2 cm (1/2 in) thick lauan plywood. Windows, doors, interior wall sheathing, and ceiling coverings were not installed during testing.

The structure was tested at six stages during construction. During Stage 1, let-in bracing was installed in both directions parallel and perpendicular to the lateral load and a portion of the second floor sheathing was omitted. The sheathing that was omitted during Stage 1 was installed for testing during Stage 2. For Stage 3 testing, the shearwall frames parallel to the direction of loading were sheathed with exterior wall siding except for areas above and below window and door openings. For Stage 4 testing, the shearwall frames perpendicular to the direction of loading were sheathed with exterior wall siding except for areas above and below window and door openings. Exterior wall siding was installed to the wall spaces above and below door and window openings on the exterior perimeter frames for Stage 5 testing. The construction of the structure remained the same for Stage 6.

During test Stages 1 through 5, each shearwall frame was loaded individually using hydraulic jack actuators at only the second floor to measure the initial stiffness of each shearwall frame. The concentrated lateral load was cycled increasing in magnitude with each cycle. During Stage 6 testing, the entire structure was loaded at once with two partially-distributed loads at the second floor using a cyclic push-pull loading that increased in magnitude with each cycle. Fifty-nine electric dial gages were used to record the displacements in both horizontal directions of points along the
second floor at the sill plates. In addition, the vertical displacements of columns with let-in bracing were recorded.

Since the load was applied to each shearwall frame individually during the first five test stages, four simultaneous equations could be solved to provide the stiffness of each wall. The researchers found that the total stiffness of the first floor walls in Stage 1 and Stage 2 were nearly the same suggesting that the larger opening in the floor diaphragm had little effect on the total wall stiffness. The total stiffness in the walls of Stage 3 was about 50% greater than that of Stages 1 and 2. This suggests that the addition of the calcium silicate siding greatly increases the lateral stiffness in the direction parallel to loading. There was negligible difference in lateral stiffness between Stages 3 and 4 suggesting that the application of the calcium silicate siding on the walls perpendicular to the direction of loading had little effect on the stiffness of the walls parallel to the direction of loading. However, it was believed that this behavior was due to very small racking deformations in the walls parallel to the direction of loading and could change at higher racking deformations. It was recommended that more tests should be conducted in this area. The total stiffness during test Stage 5 was about 10% to 15% greater than that of test Stage 4 indicating that the application of sheathing materials above and below the door and window openings increases the wall stiffness. During the entire structure testing of Stage 6, local failure of the house occurred as a result of the buckling of a let-in brace on an interior shearwall frame at an applied structure load of about 32,000 lbs (142 kN). From measurements of the second floor diaphragm deformations, it was concluded that the in-plane rotation was very small.
**Study by Yasamura (1988)**

Yasamura et al. (1988) investigated the lateral resistance of a three-story woodframe house. A revision of Japanese Building Law in 1987 permitted woodframe buildings to be constructed three stories high. The main objective of the experiment was to determine if three story woodframe houses were structurally safe when subjected to lateral loads. Another objective was to obtain experimental data for use in calculating the horizontal resistance of the structure. The shear resistance of each story was compared to theoretical calculations based on non-linear load-slip relations of a nail joint.

The three-story house was 7.28 m (23.9 ft) wide by 9.1 m (29.9 ft) long (direction of loading). In the longitudinal direction, the two exterior shearwalls at each floor were sheathed with 9.5 mm (3/8 in) thick plywood and 12 mm (1/2 in) thick gypsum wallboard. The interior shearwalls located at the center of each floor were sheathed with 12 mm (1/2 in) thick gypsum wallboard on both sides. For all wall sheathings, nails were spaced at 10 cm (4 in) around the perimeter of panels and at 20 cm (8 in) to intermediate supports. The sub-floor for each floor was sheathed with 12 mm (1/2 in) thick lauan plywood. The roof was sheathed with 9.5 mm (3/8 in) thick lauan plywood.

The structure was tested under three different sheathing configurations during construction. During the testing of Specimen A, the longitudinal walls were sheathed, but the sub-floor and the transverse walls were not sheathed. The longitudinal walls and the sub-floor were sheathed during the testing of Specimen B. The longitudinal
walls, transverse walls, and sub-floor were all sheathed during the testing of Specimen C.

The structure was loaded using three hydraulic jacks on the top of the third story at the two exterior shearwalls and the interior shearwall. Electric transducers measured horizontal and vertical displacements of each story and tension forces in each holdown bolt. In the testing of Specimen A, the load was applied monotonically producing equal displacements at each of the three loading points. Specimens B and C were tested with an increasing cyclic load at each of the three shearwalls. In these last two tests, the load applied to the interior wall was one and a half times the load applied to the exterior shearwalls. In addition, a rotation mass generator at the roof was used for forced vibration tests of Specimens B and C.

As a result of the experimental testing, the researchers discovered that the shear resistance of the north longitudinal wall was approximately one and a half times the shear resistance of the south longitudinal wall due to a larger number of openings on the south wall. However, this difference in shear resistance had very little effect on the torsional deformation of the structure since the transverse walls had sufficient stiffness to resist torsional deformation. The shear deformations were relatively small in the horizontal diaphragms suggesting that they should be considered rigid. The researchers found that the horizontal load was distributed approximately in proportion to the length of shearwalls without openings. Much of the failure in all of the specimens occurred on the second story, particular in the south and middle walls. The exterior plywood and interior gypsum wallboard buckled on the compression side around openings and was torn on the tension side around openings.
Through the forced vibration tests, several important patterns were noticed in the natural frequencies of the structure. After subjecting Specimen C to horizontal loads and causing damage, the fundamental natural frequency decreased from 5.8 Hz to 3.1 Hz. The finished house with all interior and exterior finish materials had a natural frequency of 4.8 Hz. In addition, the application of sheathing to the transverse walls resulted in an increase in the torsional natural frequency from 4.8 Hz to 8.8 Hz.

The experimental results obtained were compared to theoretical results using Tuomi’s racking model. This model predicts a load-deformation relationship for shearwalls using the height and width of the shear panel, nailing, load-slip, and shear strain of the panel. A least squares method was used to develop load-deformation relationships for the experimental results of the plywood and gypsum wallboard shearwalls. It was found that the theoretical horizontal resistance predicted by Tuomi’s racking model exceeded the experimental horizontal resistance. However, if a modification was made to the load-slip relation used in Tuomi’s racking model, the theoretical racking resistance was very close to the experimental racking resistance.

**Study by Carydis and Vougioukas (1989)**

Carydis and Vougioukas (1989) tested a two-story timber frame construction house using a 6-degree of freedom earthquake simulator at National Technical University of Athens, Greece. The purpose of this experimental study was to demonstrate that wood houses constructed according to the Timber Frame Construction (T.F.C.) method were adequately resistant to earthquakes.
The two-story test structure was 3.6 m (11.8 ft) wide by 3.6 m (11.8 ft) long with a first story height of 2.9 m (9.5 ft) and a second story height of 2.6 m (8.5 ft). The Canadian Code of T.F.C. was followed for construction and the Greek Code was followed for dead and live loads. The exterior walls of the house were covered with asbestos cement over a bituminous paper with a chickennet metal. On two adjacent walls, plastic fibers were used to reinforce the cement plaster. In order to provide an adequate dead and live load in the structure, sacks filled with sand were placed on the second floor and roof.

The main shock of the Kalamata earthquake was used to excite the structure in the three principle directions simultaneously. The Kalamata earthquake struck the city of Kalamata on September 13, 1986 with a magnitude of 6.2 on the Richter scale. The test structure was tested with 40 repetitions of the Kalamata earthquake. The structure was instrumented with a few accelerometers to record the response.

Overall, the test structure performed well during the testing. After approximately 3 to 4 repetitions of the earthquake, nails at the base became loose and required repair. The plaster on the exterior cracked, but this cracking did not correspond to any damage to the wood substructure. During repetitions 20 to 40, the cracks did not significantly enlarge. After the 40th repetition of the Kalamata earthquake, the largest crack in the plaster had a width of the order of 2 mm (1/16 in). However, after the 20th repetition, a beam in the second floor failed at the location of an existing knot due to relatively high vertical accelerations.

After the 1st repetition of the earthquake, the measured natural period of the structure was 0.18 seconds in the longitudinal and transverse directions, and 0.16
seconds in the vertical direction. After the 20\textsuperscript{th} repetition, the natural period was 0.21
seconds in the longitudinal direction, 0.20 seconds in the transverse direction, and 0.17
seconds in the vertical direction. After the last repetition, the natural period was 0.22
seconds in the longitudinal and transverse directions, and 0.17 seconds in the vertical
direction. The maximum acceleration of various parts of the structure was of the order
of 2 g. The damping in the structure varied from repetition to repetition. The damping
of the structure was around 17\% of critical during the 15\textsuperscript{th} repetition of the earthquake.

**Study by Phillips et al. (1993)**

Phillips et al. (1993) tested a full-scale single story wood-framed structure
subjected to lateral loads in 1990. The main objective was to determine the load-
sharing characteristics of different building components through measured internal
forces and displacements. This experimental study determined how a horizontal
diaphragm affects the distribution of the load into shearwall elements. In addition, the
study evaluated the hysteretic response of the test structure under cyclic lateral load.
The stiffness of the wood shearwalls with different sheathing materials was also
determined.

The 16-ft (4.9 m) wide (direction of loading) by 32-ft (9.8 m) long single-story
test structure was constructed in a laboratory. The floor of the test structure was
constructed with two-by-ten joists spaced at 16 in (0.4 m) with 5/8 in (16 mm) thick
plywood sheathing. Four 16-ft (4.9 m) transverse shearwalls divided the house into
three rooms. Each shearwall was designed to produce a different lateral stiffness by
using different sheathing materials and varying sizes of door and window openings.
The shearwalls were framed with two-by-fours spaced at 16 in (0.4 m) with a single two-by-four sill plate and a double two-by-four top plate. The exterior west and east shearwalls were sheathed with 19/32 in (15 mm) thick T1-11 exterior siding on the outside and ½ in (13 mm) plywood on the inside. The interior west shearwall was sheathed with ½ in (13 mm) thick gypsum wallboard on both sides. The interior east shearwall was sheathed with ½ in (13 mm) thick plywood on both sides. The longitudinal walls were constructed similarly to the shearwalls with 19/32 in (15 mm) thick T1-11 exterior siding on the outside and ½ in (13 mm) thick gypsum wallboard on the inside. The roof was constructed with nine manufactured trusses spanning the structure in the longitudinal direction. The roof was sheathed with ½ in (13 mm) thick exterior plywood and the ceiling was sheathed with ½ in (13 mm) gypsum wallboard.

Two servo-controlled hydraulic actuators were used to apply concentrated loads at the top of each of the four shearwalls. A steel I-beam was used to divide the load from each hydraulic actuator into point loads at the top of two shearwalls. The rate of loading and unloading of the walls was 1.78 kN/min. (400 lb/min.) as specified by ASTM E72. The structure was tested in four stages during the construction process. Stages I and II tested the racking resistance of the transverse shearwalls prior to the construction of the longitudinal walls or roof diaphragm. During Stage I, the four shearwalls were tested with sheathing on one side only. During Stage II, the shearwalls were tested with sheathing installed on both sides. The four transverse shearwalls and the longitudinal walls were constructed for Stage III testing. During Stage IV, the complete structure with all walls and roof diaphragm was tested with all
walls and roof diaphragm. Stage IV test loads were considerably larger to represent overload conditions.

The structure was instrumented with four load cells at each point of load application. Each of the four shearwalls was instrumented with four horizontal and four vertical load cells to measure the internal forces at the base of the walls. Linear variable differential transducers (LVDTs) measured the gross displacement at the top of each wall, the sill plate slippage, and the sill plate uplift for each shearwall.

During Stage I and II, load-deformation relationships were developed for each shearwall. The resulting relationships showed that the stiffness of the shearwalls was additive with additional layers of sheathing. The hysteretic loops exhibited a pinching behavior common in timber shearwalls. During Stages III and IV, the loads were first applied to adjacent pairs of shearwalls and then to all four shearwalls simultaneously. This loading sequence allowed the load sharing characteristics of the structure with and without the roof diaphragm to be determined. During Stage III, applied loads were not transferred to unloaded walls without the roof diaphragm. The exterior shearwalls carried a larger percentage of the total applied load compared to the interior shearwalls due to the corner connections. During Stage IV, the roof diaphragm transferred some of the load into the unloaded walls. During these tests, the longitudinal walls carried up to 23% of the applied load, but this percentage decreased at higher levels of loading. The experimental results of the load distribution were compared to calculations following the 1991 UBC. From these calculations, it was concluded the roof diaphragm behaved more like a rigid diaphragm than a flexible diaphragm.
Study by Tanaka et al. (1998)

Between November and December 1995, Tanaka et al. (1998) tested a full-scale, two-story, post and beam woodframe house using a shake table. The 1995 Kobe Earthquake caused significant damage to post and beam woodframe houses in Japan resulting in loss of life. From the testing of this house, the safety and seismic performance of woodframe housing in Japan could be assessed. In addition, the influence of nonstructural sheathing materials on the dynamic response of the structure could be evaluated. An analytical model was developed in an effort to predict the experimental response.

The two-story house was 7.28 m (23.9 ft) wide by 7.28 m (23.9 ft) long with a total floor area of both floors of 53.0 m² (570 ft²). Each story was approximately 2.95 m (9.7 ft) high and the total height of the structure at the ridge was 7.54 m (24.7 ft). The structure was designed using a seismic shear coefficient of 0.28. The exterior of the building was sheathed with siding and the interior was covered with gypsum wallboard. Diagonal braces were used to resist the lateral forces in the shearwalls. The total length of shearwalls in the test structure was approximately 1.5 times the required length of shearwalls established by the building code. If the structure were to be simplified as a two-degree-of-freedom system, the mass at the roof would be 9.0 tons and the mass at the second floor level would be 9.3 tons.

The test structure was excited using a 15 m (49.2 ft) wide by 15 m (49.2 ft) long shake table capable of motion in one horizontal and one vertical direction. The test structure was excited using the 1995 Kobe Earthquake recorded at the Japan Meteorological Agency (JMA) Kobe station and the 1940 El Centro Earthquake with
an amplitude scale factor of 1.5. For both earthquakes, the North-South component and the vertical component of the motion were used to excite the structure. In addition to ground motion testing, sweep frequency testing was also performed with a constant acceleration of 30 gal (0.03 g) to measure the natural frequencies of the house. The structure was tested during three phases of construction with different sheathing materials for a total of 8 experiments with different input motions. For Phase 1, the exterior siding, interior gypsum wallboard, and structural frame were installed for testing. The exterior siding was removed for testing during Phase 2. For Phase 3, only the structural frame with diagonal braces was tested. Damage to the gypsum wallboard was repaired between Phases 1 and 2 and the diagonal braces were replaced between Phase 2 and 3. The test structure was instrumented with accelerometers, displacement meters, and strain gages.

By removing sheathing materials with each phase of testing, it was found how this affected the damage in the structure. During the first phase of testing with the JMA Kobe ground motion and the El Centro ground motion, damage was limited to minor cracking in the exterior siding and gypsum wallboard around window openings. During Phase 2, cracks again appeared in the gypsum wallboard around window openings. Three of the fourteen diagonal braces on the first floor failed by buckling or by tensile failure. There was no damage to the second floor diagonal braces. Ten of the fourteen first floor diagonal braces failed during Phase 3. In addition, all eight of the second floor diagonal braces failed during this phase of testing. These trends in damage to the structural frame suggest that the nonstructural finish materials resist a significant portion of the lateral forces in the structure.
During Phase 1 of testing, the maximum acceleration recorded at the roof level was 2123 gal (2.16 g), equal to 2.6 times the peak input acceleration. The maximum acceleration recorded during Phase 2 was smaller than Phase 1. The acceleration response during Phase 3 was very small because the diagonal braces failed, which greatly reduced the lateral stiffness of the structure. By using the same JMA Kobe ground motion for all three phases of testing, the researchers were able to compare the story drift levels for each phase. For Phase 1, the first story drift was 1/30 rad (3.3%) and the second story drift was 1/89 rad (1.1%). The first story drift was 1/15 rad (6.7%) and the second story drift was 1/40 rad (2.5%) for Phase 2. During the last phase of testing, the first story drift was 1/18 rad (5.6%) and the second story drift was 1/17 rad (5.9%). These drift levels suggest the exterior siding is very effective in resisting lateral deformation. The maximum total base shear during testing was about 20 tons representing a seismic coefficient between 1.0 to 1.1. The maximum axial compression was 3 to 4 tons and the maximum axial tension load was 1 to 2 tons in the diagonal braces. The axial load in the braces was considerably larger during Phase 3 than during Phase 1 because the gypsum wallboard and exterior siding resisted a significant portion of the load during Phase 1. From these results, it was concluded that the nonstructural sheathing materials added considerable stiffness to the structure.

An analytical model was developed to predict the experimental response. A simple two degree-of-freedom shear model was used with lumped masses at the second floor and roof levels. Bilinear hysteresis loops were used for the shearwall elements. The analytical model matched the experimental hysteric loops well. In
addition, the story drift predicted by the analytical model at the first and second floor matched the experimental drift.

**Study by Kohara and Miyazawa (1998)**

Following the 1995 Hyogo-ken Nanbu Earthquake in Japan, a joint research group composed of Kogakuin University, Tokyo University, and others tested six two-story woodframe houses using a shake table. In this study, the Kohara and Miyazawa (1998) discussed the testing and results of two of the structures: Type B and Type F. The main objective of these tests was to determine the damage to the structures resulting from specific ground motions used on the shake table. Another objective was to quantify the dynamic behavior of the structures including the natural frequencies, displacements, and accelerations. Lastly, it was planned to investigate the hysteresis loops of the shearwalls in the structures.

Both two-story test structures were 11.83 m (33.8 ft) long by 7.28 m (23.9 ft) wide with story heights of 2.9 m (9.5 ft). The total floor area of each structure was 133 m² (1432 ft²). For both structures, lateral forces were resisted by varying amounts of diagonal braces and plywood sheathing during different phases of testing. The interior wall surfaces of both structures were sheathed with gypsum wallboard. The exterior wall surfaces of structure Type B were covered with mortar stucco. The exterior wall surfaces of structure Type F were sheathed with siding. The roofs of both structures were covered with roofing tiles.

The two test structures were tested with various construction configurations using several input motions. The structures were tested using a sine wave sweeping
frequency motion, the Japan Meteorological Agency (JMA) 1995 Kobe Earthquake, and the 1940 El Centro Earthquake with a scalefactor of 1.5. In addition, microtremor measurements were made for each experiment to determine the natural frequencies of the structures. The longitudinal direction of both test structures was excited with the North-South component of these ground motions. The transverse direction of both test structures was excited with the East-West component of these ground motions. Both structures were excited with the vertical component of the ground motions.

Structure Type B was tested during five different phases for a total of eleven experiments. For Phase 1, the structure was tested with diagonal braces, mortar stucco, and gypsum wallboard. The sill plate anchor bolts were fastened and one-third of the mortar stucco was removed for Phase 2. One-third of the gypsum wallboard was removed and the sill plate anchor bolts were not fastened for Phase 3. For Phase 4, the sill plate anchor bolts were fastened, one-third of the diagonal braces were removed, and an additional one-sixth of the gypsum wallboard was removed. The sill plate anchor bolts were again fastened and the majority of the diagonal braces were removed and replaced with plywood sheathing for Phase 5.

Structure Type F was tested during five different phases for a total of ten experiments. For the first phase, diagonal braces, siding, and gypsum wallboard was installed throughout the test structure. Eccentricity was introduced in the test structure by making an opening on the south side of the building and rearranging the diagonal braces to alter the stiffness of some of the shearwalls for Phase 2. For Phase 3, the exterior wall siding was removed from the first floor walls and the sill plate anchor bolts were fastened. The gypsum wallboard on the interior first floor walls was
removed for Phase 4. For Phase 5, most of the diagonal braces were removed and replaced with plywood sheathing and the holdown anchors were fastened.

At the start of experimental testing on the structures, the natural frequencies were 6.49 Hz and 6.05 Hz for structure Type B and Type F, respectively. As a result of removing diagonal braces, mortar stucco, and gypsum wallboard over different phases of testing and cumulative damage effects, the natural frequencies of the structures decreased with each test. It was found that when the natural frequency of the structure was 4 to 5 Hz, extensive damage occurred to the mortar stucco or exterior siding board and to the gypsum wallboard. In addition, it was found that damage occurred to the structure’s frame when the natural frequency of the structure was around 3 Hz.

From the first phase of testing of both structures with the JMA Kobe earthquake, it was found that the input acceleration was magnified 1.3 to 1.8 times in the structure. The structures had first story displacements between 1/52 rad and 1/120 rad (0.8% to 1.9% drift) during Phase 1. By removing one-third of the mortar stucco from structure Type B, the input acceleration was magnified 2.0 to 2.3 times in the structure with first story displacements between 1/28 rad and 1/46 rad (2.2% to 3.6% drift). After removing the exterior siding from structure Type F, the input acceleration was magnified 2.2 to 2.4 times with first story displacements between 1/23 rad and 1/29 rad (3.4% to 4.3% drift). These results suggest that both the stucco mortar and exterior wall siding were both effective in limiting the response of the structures.

In structure Type B, it was found that the diagonal braces resisted between 7% and 17% of the total base shear while the stucco mortar resisted between 21% and
47% of the base shear. In structure Type F, the diagonal braces resisted between 29% and 54% of the total base shear. From the hysteresis loops obtained, it was found that walls with mortar stucco from structure Type B had a much higher stiffness than those with exterior siding from structure Type F. However, the stiffness of the walls with mortar stucco decreased over the phases of testing due to cracks. The stiffness of the walls with exterior siding remained constant during testing.

**Study by Ohashi et al. (1998)**

In 1996, Ohashi et al. (1998) tested three woodframe houses subjected to ground motions on a shake table. Although three woodframe houses were tested, only the single-story structure was reported in this paper. Since woodframe houses suffered extensive damage during the 1995 Hyogoken-Nanbu (Kobe) Earthquake in Japan, researchers wanted to determine the seismic safety of these structures. In addition, they wanted to quantify the effects of nonstructural finish materials on the stiffness of the structure. Another objective was to examine the woodframe house with configurations in which the stiffness of the house was symmetric and eccentric.

The test structure was 5.4 m (17.7 ft) long by 3.6 m (11.8 ft) wide and 2.9 m (9.5 ft) high. Although the test structure was a single story, it was equivalent to the first story of a two-story structure. The post and beam structure was constructed with 120 mm (4 ¾ in) by 120 mm (4 ¾ in) columns at the corners with 45 mm (1 ¾ in) by 90 mm (3 ½ in) diagonal braces. The north and south walls of the house had a single door opening at the center and the east and west walls each had two symmetrically placed window openings. The second floor of the house was sheathed with plywood.
In some phases of testing, the interior surface of the walls was sheathed with gypsum wallboard and the exterior surface of the walls was sheathed with cement fiberboard siding. The sill of the house was fastened to the steel base with anchor bolts. In addition, the corner columns were attached to the base with holdown anchors. Additional weight was added to the floor in order to create a seismic shear coefficient of 0.2.

The house was tested during four stages of construction. For Phase 1, the structure (without gypsum wallboard or cement fiberboard siding) was tested with a low supplemental dead and live weight. The structure was tested using the full dead and live weight in both standard and eccentric braced conditions for Phase 2. Each of the four walls was constructed with four diagonal braces for the standard braced condition. For the eccentric braced condition, the North and West walls were constructed with six braces in each wall and the South and East walls were constructed with two braces in each wall. For Phase 3, the frame structure with gypsum wallboard was tested in both standard and eccentric braced conditions. The cement fiberboard siding was installed for Phase 4 and tested in only the standard braced condition.

The structure was tested using the JR-Takatori record from the 1995 Kobe Earthquake with the North-South component, East-West component, and the vertical component. All four phases were tested using the JR-Takatori ground motion scaled to a maximum velocity of 25 cm/s, equivalent to one-fifth of the full-scale recorded motion. In addition, one phase was tested using the JR-Takatori ground motion scaled to a maximum velocity of 50 cm/s. Two phases were tested using the full-scale JR-Takatori ground motion with a maximum velocity of 135 cm/s in the North-South
direction. In addition, micro-tremor measurements were made before and after each phase of testing to measure the natural frequencies of the structure. Random motion excitation tests and sine wave excitation tests were preformed to verify the natural frequencies.

Following the final phase of testing with both gypsum wallboard and exterior siding installed, no damage was found after the 50 cm/s excitation. After the first full-scale excitation, cracks formed in the gypsum wallboard around the corners of openings. After the second full-scale excitation, the cracks in the gypsum wallboard became larger and some of the exterior siding began to fall down. However, there was no damage to the structural frame and braces. Accelerations were recorded up to 1000 gal (1.02 g) at the floor level in the North-South direction representing a response magnification factor of 1.3. From the natural frequency testing of the structure phase by phase, the stiffness of the structure increased as the gypsum wallboard and exterior siding were installed. As a result, the percentage of the total base shear resisted by the diagonal braces decreased as the additional finish materials were applied. During Phase 3 with gypsum wallboard installed, the diagonal braces resisted between 30% and 50% of the total base shear. During Phase 4 with both gypsum wallboard and exterior siding installed, the diagonal braces resisted between 10% and 20% of the total base shear. In addition, the tension forces in the holdown anchors increased as the finish materials were applied.

An analytical model using a bi-linear and slip hysteresis was developed. Newmark’s beta method was used for integration of the ground motions. The stiffness and the yield displacement for the hysteresis model were determined from the force-
deformation relationships measured in the experimental testing. The researchers concluded this method could be used to predict the seismic performance of a woodframe structure since the results were close to the measured response of the structure.

**Study by Yamaguchi and Minowa (1998)**

Following the 1995 Kobe Earthquake in Japan, Yamaguchi and Minowa (1998) tested timber shearwalls using a shake table. Only bearing walls were tested because of ease in changing the walls as compared to changing a whole structure between phases of testing. The main objective was to compare dynamic hysteresis loops developed in this experiment with previously developed static hysteresis loops of timber shearwalls. Since most analytical computer models use static hysteresis loops for shearwalls, it was important to know the relation between static and dynamic hysteresis loops. In addition, a collapse analysis using conservation of energy was performed.

The test specimens were 3.64 m (12 ft) long by 2.94 m (9.6 ft) high with a 1.82 m (6 ft) wide opening at the center of the wall. The bearing walls were framed using a continuous beam across the top of the wall and a continuous sill at the base of the wall. Four 105 mm (4.1 in) by 105 mm (4.1 in) columns supported the header beam and were connected to the beam and sill using vertical bolts. The wall was sheathed with 9 mm (3/8 in) thick plywood sheets 0.91 m (3 ft) wide by 2.73 m (9 ft) tall. The sheathing was nailed to the framing using N50 nails (2.7 mm (0.106 in) diameter by 50 mm (1.97 in) long) spaced at 150 mm (5.9 in). The shearwalls were placed in a
moving steel frame that prevented the shearwalls from taking any vertical load yet still allowed the walls to resist lateral inertia loads from weights on the moving steel frame.

A 15 m (49.2 ft) long by 14.5 m (47.6 ft) wide earthquake simulator at the National Institute of Earth Science and Disaster Prevention in Japan was used to excite the shearwalls with the Japan Meteorological Agency (JMA) Kobe North-South ground motion. Three different specimens with seismic shear coefficients of 0.3, 0.4, and 0.5 were used for testing. The seismic shear coefficient is defined as the lateral force in a wall when the tilting angle of the wall is 1/120 rad (0.8% drift) divided by the weight on the wall. Servo accelerometers were used to measure accelerations at the shake table and at the top of the wall. In addition, laser displacement meters were used to measure displacements at the top of the walls.

During the testing of the three specimens with the JMA Kobe ground motion, the shearwalls with seismic shear coefficients of 0.3 and 0.4 collapsed during testing. Force-deformation relationships were developed for each of the three shearwalls and then compared to the static monotonic force-deformation relationships developed previously. For the first specimen with a seismic shear coefficient of 0.3, the dynamic hysteresis matched well with the static hysteresis until a tilting angle of about 1/120 rad (0.8% drift). However, after a tilting angle of 1/120 rad (0.8% drift), the tilting angle of the static hysteresis increased rapidly due to creep effects in the timber. The maximum strength of the shearwall during the dynamic test was 114% of the maximum strength during the static test. In addition, the displacement at the maximum strength for the dynamic test was 50% of the displacement at the maximum strength for the static test. A static yield point was not observed on the dynamic hysteresis, but
a dynamic yield point was observed just prior to the maximum strength on the dynamic hysteresis. From these results, it was concluded that a shearwall subjected to dynamic loads has more strength but less ductility than a shearwall subjected to static loads.

Since the shearwall with a seismic shear coefficient of 0.4 collapsed and the shearwall with a seismic shear coefficient of 0.5 did not, this type of shearwall will collapse with this ground motion with a seismic shear coefficient between 0.4 and 0.5. A method for predicting the collapse of shearwalls based on the experimental data and conservation of energy was proposed. This method first involves determining the equivalent natural period of a shearwall based on the elastic stiffness. Using a response spectrum for the ground motion, the spectral displacement and the spectra acceleration could be determined using the equivalent natural period. The potential energy at collapse can be determined from one dynamic hysteresis loops since potential energy is not dependent on the seismic shear coefficient. If the potential energy is more than the required energy, the shearwall is predicted to not collapse. If the potential energy is less than the required energy, the shearwall is predicted to collapse. This method was used to test three additional shearwalls using the 1940 El Centro ground motion. All three of these tests accurately predicted whether the shearwalls would collapse.

**Study by Seo (1999)**

In 1998, Seo et al. (1999) tested two one-quarter-scale post and beam woodframe house models using a shake table. The main objective of this experiment was to determine the maximum peak ground acceleration that an ancient Korean commoner’s
house located in areas of low to moderate seismicity could sustain without collapse. The first model was subjected to an earthquake record from a rock site and the second to an earthquake record from a soft soil site. The ground motions were scaled such that the peak ground acceleration level increased incrementally until the test models experienced severe damage or failure. In addition, the natural frequency and the damping in the test models were measured.

The two identical single-story one-quarter-scale models were 1.8 m (5.9 ft) long by 0.9 m (3.0 ft) wide by 0.7 m (2.4 ft) high. The models were fabricated using fresh pine lumber with tenons at beam-column joints and dovetails at column-cross member joints. The roof was constructed with round timbers scaled to a diameter of 22.5 mm (7/8 in) at 75 mm (3 in) on center. To simulate the weight of mud plaster and straw thatches typically for the roof of these houses, 25 mm (1 in) thick steel plates were used to produce a total roof mass of 930 kg (2050 lbs). The ancient Korean commoner’s houses were typically supported on rock foundation blocks at the columns. For the experiment, a ball bearing was used to create a hinge condition at the bottom of each column. Each model was instrumented with nine accelerometers and six Linear Variable Displacement Transformers (LVDTs) to measure the response at the top of two corner columns and the center column.

The two models were tested on a 4 m (13.1 ft) wide by 4 m (13.1 ft) long six degree-of-freedom shake table. The electro-hydraulic servo-control system of the shake table was capable of producing maximum horizontal accelerations of 1.5 g and maximum vertical accelerations of 1.0 g with a maximum specimen mass of 30,000 kg (66.1 kips). In this experiment, the first model was subjected to the Nahanni
Earthquake recorded at a rock site in eastern Canada in 1985. The second model was subjected to the Imperial Valley Earthquake of October 1979 recorded at a soft soil site in the western United States. Two horizontal components and one vertical component of the earthquake ground motion were used in the shake table testing of both models. For the testing of Model 1 with the rock site ground motion, the peak horizontal ground acceleration was increased from 0.1 g to 0.6 g in increments of 0.1 g. For the testing of Model 2 with the soft soil site ground motion, the peak horizontal ground acceleration was increased from 0.05 g until failure in increments of 0.05 g. The peak vertical ground acceleration was scaled to two-thirds of the peak horizontal ground acceleration for each of the tests. In addition to exciting the two models with the earthquake ground motions, random white noise was inputed to the shake table to measure the natural frequency and damping of the models.

From the random white noise tests, the natural frequencies of Model 1 were 3.32 Hz and 3.52 Hz in the longitudinal and transverse directions, respectively. The natural frequencies of Model 2 were 3.32 Hz and 4.29 Hz in the longitudinal and transverse directions, respectively. Since both models were one-fourth scale, a prototype structure would have natural frequencies one-half of those recorded in the models. The difference in natural frequencies between the two models in the transverse direction was attributed to the method of fabrication of the models and to the carpenter skill. The modal damping ratio of both models was seven percent in both directions.

The first model was tested up to a peak horizontal ground acceleration of 0.6 g with a ground motion from a rock site. The peak acceleration of the model was
significantly less than the input peak acceleration of the ground motion, especially at higher levels of the peak ground acceleration. The peak vertical acceleration of the structure was slightly more than the input peak vertical acceleration of the ground motion. Acceleration and displacement responses of the structure in the longitudinal direction were larger than the transverse direction due to differences in energy content of the ground motion. The second model was tested up to a peak horizontal ground acceleration of 0.25 g with the ground motion from a soft soil site. The trends of the acceleration and displacement responses were similar to those found in Model 1. However, Model 2 failed at a lower peak horizontal ground acceleration of 0.25 g in the longitudinal direction. It was noted that the peak ground acceleration at failure might have been low due to cumulative damage effects during the incremental testing of the models.

The experimental results of both models were compared to non-linear dynamic analyses that used the modified Double-Target model. The researchers obtained analytical model characteristics such as the stiffness and damping ratio from the random white noise tests. The two models were idealized as single-degree-of-freedom structures with lumped masses. For Model 1, the analytical model was run using an input peak horizontal ground acceleration of 0.6 g in both horizontal directions. The analytical model predicted smaller accelerations and displacements than the experimental test model. It was believed that this may have been a result of not using the vertical component of the ground motion in the analytical model or cumulative damage effects in the experimental model. For Model 2, the analytical model was run using a input peak horizontal ground acceleration of 0.08 g in both horizontal
directions. The analytical model predicted smaller accelerations and larger peak displacements. This discrepancy may have been a result of the stiffness of the transverse frames used in the analytical model. Overall, the modified Double-Target model was able to simulate the non-linear dynamic response of a post and beam house with tenon joints.

**Study by Takahiro et al. (2000)**

There were a significant number of woodframe houses with large eccentricity that were either damaged or destroyed following the 1995 Hyogo-Ken Nanbu (Kobe) Earthquake in Japan. The damage to these structures was caused by a lack of effective shearwall length, eccentricity of the shearwall configuration, and inadequate performance of seismic fasteners at the end of braces and columns. As a result, Takahiro et al. (2000) conducted an experimental investigation of Japanese woodframe houses with large eccentricity using a shake table. Eight different one-story specimens were tested with different eccentricities and three additional specimens were tested each with a different stiffness of the roof diaphragm.

Each full-scale specimen was 5.55 m (14.9 ft) long by 3.64 m (11.9 ft) with a story height of 2.95 m (9.7 ft). Corner columns and columns adjacent to wall openings were 105 mm (4.1 in) by 105 mm (4.1 in) in cross section. Each column was fastened to the sill plate using metal fasteners. The shearwalls were sheathed with 9 mm (3/8 in) thick plywood and fastened with 2.87 mm (0.112 in) diameters nails at 200 mm (7.9 in) on center. The roof was constructed with 45 mm (1.8 in) by 60 mm (2.4 in) joists at 455 mm (17.9 in) on center. The joists were covered with 180 mm (7.1 in)
wide by 13 mm (0.5 in) thick planks with two nails into each joist. The roof was sheathed with 12 mm (1/2 in) thick plywood with 2.87 mm (0.113 in) diameters nails at 150 mm (5.9 in) on center.

For each of the test specimens, a series of excitations were inputted on the structure. A sine-sweep input with an acceleration of 30 gal (0.03 g) was used to measure the fundamental natural frequency of each test specimen. A sine wave with an input acceleration of 200 gal (0.20 g) at the fundamental natural frequency of the test specimen was then used to excite each test specimen. The JMA Kobe North-South ground motion with a peak ground acceleration of 818 gal (0.83 g) followed the sine wave excitations. Lastly, the JMA Kobe North-South ground motion with an amplitude-scaling factor of 1.5 was used to excite each test specimen. The test specimens were instrumented with 7 accelerometers in the direction of shaking along with 3 accelerometers perpendicular to the direction of shaking. Displacement sensors were also installed at locations close to the accelerometers. In addition, strain gages were attached to the plywood of the shearwalls and roof diaphragm, columns, and on bolts for holdowns.

The fundamental natural frequency of the test specimens ranged from about 3 Hz to 7 Hz. It was found that the fundamental natural frequency of each test specimen deteriorated by 0.5 Hz to 1.5 Hz following the sequence of tests performed. Significant torsional behavior was evident in the test structure with large eccentricity as shown by the deformation and acceleration response of the test structure. There was a large rotational component of deformation for the eccentric test specimens. In addition, the acceleration distribution across each eccentric test specimen was nearly
triangular. Significant deformation of the roof diaphragm was noted for the three specimens with a low in-plane stiffness of the roof diaphragm.

For each of the shake table tests, hysteresis loops for each shearwall were developed using the shear force in each shearwall (measured using the strain gages on the plywood). These results showed that each exterior shearwall frame deteriorated equivalently for the test specimen without eccentricity. However, stiffness degradation occurred more in the shearwall with the large opening for the eccentric test specimens. It was found that the distribution of shear force in each exterior shearwall was independent of the eccentricity for the test specimens with a low in-plane stiffness of the roof diaphragm. However, the distribution of shear force in each exterior shearwall was highly dependent on the eccentricity of the test specimen if the in-plane stiffness of the roof diaphragm was high. From these tests, it was found that the dynamic distance of eccentricity for the calculation of torsional stress should be 1.5 times that of the static distance of eccentricity.

2.2 Motivation for Research

As a result of damage to residential woodframe structures from recent earthquakes in the United States, more shake table testing is required for these structures. There is a lack of full-scale testing of houses framed with two-by-four walls and sheathed with plywood or Oriented Strand Board (OSB). This shake table testing should examine the common construction methods used in the United States. Based on shake table testing in Japan, this testing should examine the influence of finish materials on the lateral stiffness of the structures. In addition, the testing should investigate the influence of window and door openings of the dynamic performance of
the structure. Previous testing shows a need for information on the distribution of forces in the full-scale structure. This report contributes in shedding some light on these issues.
Chapter 3  Test Structure and Testing
Objectives

3.1 Description of Test Structure

The test structure considered in Task 1.1.1 of the CUREe-Caltech Woodframe Project represents a simplified full-scale two-story single-family house incorporating several characteristics of recent California residential construction. The emphasis was put on simpler construction for which the results could more easily be interpreted, rather than incorporating complicated geometry features such as floor cantilevers and roof offsets. The architectural and structural plans for the final configuration (Phase 9 and 10) of the test structure are shown in Appendix A. The Managers of Element 3 of the CUREe-Caltech Woodframe Project prepared these plans in collaboration with the
Investigators of Task 1.1.1 (Cobeen, 2000). Figure 3.1 shows plan views of the first story and second story with the major structural components incorporated in the design. Similarly, Figure 3.2 shows elevations of the east, west, and north and south walls with the major structural components.

The footprint of the structure is 16 ft x 20 ft and was anchored to the shake table such that shaking occurred along the short dimension of the structure (north-south direction). The construction was at full scale, however the plan dimensions of the structure are smaller than would be of a typical residence due to the restrictions of the shake table. An attempt was made to maintain the character of typical shearwalls within the smaller plan dimensions. The lateral load resisting system of the test structure parallel to the shaking direction (east and west wall elevations) consisted of exterior shearwalls. For the final configuration, the openings of these walls have been
Figure 3.2: Elevations of test structure showing major structural components
designed in an attempt to reproduce the torsional eccentricity that would be induced by
a large garage door opening on one side of a residence. The lateral load-resisting
shearwalls also supported the gravity loads together with first-floor interior bearing
wall and a glue-laminated beam at the second floor. In the north and south wall
elevations, exterior shearwalls provided a torsional restraint to the test structure. The
openings in these walls, which were perpendicular to the shaking direction, have been
kept conservative and symmetric to aid in simplifying the interpretation of the experimental results. The design was prepared based on the engineering design provisions of the 1994 edition of the Uniform Building Code (ICBO, 1994) and common design practices in California. The design assumes a seismic zone 4 and an $R_w$ factor of 6 (shearwalls with wood structural panel and a bearing wall system). In keeping with the higher shearwall aspect ratios permitted by the 1994 UBC, the design aspect ratios go up to 2.7. The 1997 UBC requirements (ICBO, 1997) for steel plate washers at anchor bolts for shearwall loads over 350 plf were incorporated. One aspect in which the residence design was different from typical construction was the specification of “dry” (19% maximum moisture content) solid sawn framing. This was intended to help reduce the variation of moisture content over the duration of the testing.

All wood structural panels were sheathed with 3/8-in thick Oriented Strand Board (OSB) and fastened to the framing with 8-penny box gun nails. The second floor structural walls were tied to the first floor walls by steel straps. The first floor walls were tied down to the foundation by steel connector devices.

3.2 UC San Diego Uniaxial Seismic Simulation Facility

The seismic tests were conducted on the uniaxial earthquake simulation system at UC San Diego featuring a 4.8-ton shake table made of an all-welded steel construction, as shown in Figure 3.3. The shake table has plan dimensions of 10 ft x 16 ft with a specimen payload capacity of 40 tons. A 90-kip dynamic-rated actuator drives the system. The bearing system consists of eight 5-in Garlock DU cylinders sliding on two stationary shafts. The usable peak-to-peak stroke is 12 in. The flow rate
of the hydraulic system allows a peak sinusoidal velocity of 40 in/s. The actuator can induce peak accelerations of 9.0 g for the bare table and 1.0 g for the fully loaded table. The workable frequency range of the simulator spans from 0 to 50 Hz.

The control system of the shake table includes an advanced, second-generation digital controller incorporating a Three-Variable-Control (TVC), together with Adaptive Inverse Control (AIC), On-Line Iteration (OLI) techniques and Resonance Canceling Notch Filters. This advanced control system allows the reproduction of earthquake ground motions with high fidelity.

Figure 3.3: Uniaxial Seismic Simulation Facility at UC San Diego
3.3 Experimental Setup

The test structure was anchored to the shake table by a rigid steel base that was bolted on the top plate of the table. The base incorporated outrigger arms in the east and west directions to accommodate the footprint of the test structure. Figure 3.4 presents a general view of the steel base mounted on the shake table.

![Figure 3.4: Steel Base on Shake Table](image)

Figure 3.5 illustrates the test structure and the steel base on the shake table. Threaded steel studs, welded to the top flange of the steel base, were used as anchor bolts for the sill plates. A 1 ½ inch-thick layer of grout was installed on top of the steel base beneath the pressure treated sill plates to represent a foundation as shown in Figure 3.6. The friction of the sill plate against the grout was similar to that of a true concrete foundation. A structural reaction wall was available near the shake table to...
Figure 3.5: Test Structure and Steel Base on Shake Table

Figure 3.6: Grout Between Steel Base and Sill Plate
perform quasi-static cyclic testing by linking the second floor of the test structure to the wall and using the shake table as a horizontal loading device.

3.4 Testing Objectives

The goals of the shake table testing of the simplified two-story house were quite extensive and were intended to provide data for use in other tasks of the CUREe-Caltech Woodframe Project (e.g., development of analytic tools, integration of results from other component and full scale tests, recommendations for code changes, etc.). To maximize the information that was gathered and learned, multiple tests were conducted at various stages of completion of the building’s lateral force resisting system.

Conducting tests at different stages of construction allowed data to be collected on many specific building configurations. Table 3.1 presents a summary list of the test phases and the corresponding test structure configurations. Comparison of the building’s response in each of these different configurations was used to establish variations in fundamental period and damping values, determine stiffness variations for different configurations of floor diaphragm construction, evaluate the effects of substantial changes in torsional restraint, and evaluate the performance of connections on the overall response of the structure. At the end of each stage of testing, damage to structural components and connections were visually inspected and recorded. Necessary repairs were made between test stages to return the structural components to their original strength and stiffness. In addition, the performance of finish materials, such as gypsum wallboard and exterior cement stucco, was visually inspected and documented. Damage observations were correlated with the force and
<table>
<thead>
<tr>
<th>Test Phase</th>
<th>Configuration of Test Structure</th>
<th>Type of Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>One story, east &amp; west fully sheathed, north &amp; south framed with openings, second floor diaphragm nailed at 50% with no blocking &amp; no adhesive</td>
<td>Quasi-Static Tests of Floor Diaphragm</td>
</tr>
<tr>
<td>2</td>
<td>One story, east &amp; west fully sheathed, north &amp; south framed with openings, second floor diaphragm nailed at 100% with no blocking &amp; no adhesive</td>
<td>Quasi-Static Tests of Floor Diaphragm</td>
</tr>
<tr>
<td>3A</td>
<td>One story, east &amp; west fully sheathed, north &amp; south framed with openings, second floor diaphragm nailed at 100% with 2 x 10 blocking &amp; no adhesive</td>
<td>Quasi-Static Tests of Floor Diaphragm</td>
</tr>
<tr>
<td>3B</td>
<td>One story, east &amp; west fully sheathed, north &amp; south framed with openings, second floor diaphragm nailed at 100% with 3 x 4 blocking &amp; no adhesive</td>
<td>Quasi-Static Tests of Floor Diaphragm</td>
</tr>
<tr>
<td>4A</td>
<td>One story, east &amp; west fully sheathed, north &amp; south framed with openings, second floor diaphragm nailed at 50% with no blocking &amp; PL400 adhesive</td>
<td>Quasi-Static Tests of Floor Diaphragm</td>
</tr>
<tr>
<td>4B</td>
<td>One story, east &amp; west fully sheathed, north &amp; south framed with openings, second floor diaphragm nailed at 100% with 3 x 4 blocking &amp; PL400 adhesive</td>
<td>Quasi-Static Tests of Floor Diaphragm</td>
</tr>
<tr>
<td>4C</td>
<td>One story, east &amp; west fully sheathed, north &amp; south framed with openings, second floor diaphragm nailed at 100% with 2 x 10 blocking &amp; PL400 adhesive</td>
<td>Quasi-Static Tests of Floor Diaphragm</td>
</tr>
<tr>
<td>4D</td>
<td>One story, east &amp; west fully sheathed, north &amp; south framed with openings, second floor diaphragm nailed at 100% with no blocking &amp; PL400 adhesive</td>
<td>Quasi-Static Tests of Floor Diaphragm</td>
</tr>
<tr>
<td>5</td>
<td>Two stories with roof, east &amp; west fully sheathed, north &amp; south sheathed with openings, second floor diaphragm nailed at 100% with no blocking &amp; PL400 adhesive</td>
<td>Quasi-Static Tests of Floor Diaphragm</td>
</tr>
<tr>
<td>6</td>
<td>Two stories with roof, east &amp; west sheathed with window and small door openings, north &amp; south sheathed with openings, second floor diaphragm nailed at 100% with no blocking &amp; PL400 adhesive</td>
<td>Quasi-Static Tests of Floor Diaphragm</td>
</tr>
<tr>
<td>6A</td>
<td>Two stories with roof, east &amp; west sheathed with window and small door openings with “wastewall” sheathing removed, north &amp; south sheathed with openings, second floor diaphragm nailed at 100% with no blocking &amp; PL400 adhesive</td>
<td>Quasi-Static Tests of Floor Diaphragm</td>
</tr>
<tr>
<td>7</td>
<td>Two stories with roof, east &amp; west sheathed with window and small door openings using perforated shearwall design, north &amp; south sheathed with openings, second floor diaphragm nailed at 100% with no blocking &amp; PL400 adhesive</td>
<td>Quasi-Static Tests of Floor Diaphragm + Frequency, Damping, and Seismic Tests of Complete Structure</td>
</tr>
<tr>
<td>7A</td>
<td>Two stories with roof, east &amp; west sheathed with window and small door openings using perforated shearwall design with anchor bolt adjacent to door openings, north &amp; south sheathed with openings, second floor diaphragm nailed at 100% with no blocking &amp; PL400 adhesive</td>
<td>Quasi-Static Tests of Floor Diaphragm + Frequency, Damping, and Seismic Tests of Complete Structure</td>
</tr>
<tr>
<td>8</td>
<td>Two stories with roof, east &amp; west sheathed with window and small door openings using conventional construction, north &amp; south sheathed with openings, second floor diaphragm nailed at 100% with no blocking &amp; PL400 adhesive</td>
<td>Quasi-Static Tests of Floor Diaphragm + Frequency, Damping, and Seismic Tests of Complete Structure</td>
</tr>
<tr>
<td>9</td>
<td>Two stories with roof, east &amp; west sheathed with window openings and small door opening on west wall and large door opening on east wall, north &amp; south sheathed with openings, second floor diaphragm nailed at 100% with no blocking &amp; PL400 adhesive</td>
<td>Quasi-Static Tests of Floor Diaphragm + Frequency, Damping, and Seismic Tests of Complete Structure</td>
</tr>
<tr>
<td>10</td>
<td>Two stories with roof, east &amp; west sheathed with window openings and small door opening on west wall and large door opening on east wall, north &amp; south sheathed with openings, second floor diaphragm nailed at 100% with no blocking &amp; PL400 adhesive, finished with exterior stucco and interior gypsum wallboard</td>
<td>Quasi-Static Tests of Floor Diaphragm + Frequency, Damping, and Seismic Tests of Complete Structure</td>
</tr>
</tbody>
</table>
deflection measurements of the building during those tests. A detailed description of each of the test phases listed in Table 3.1 is provided in the next chapter.

The primary objective of this shake table testing was to measure and quantify the building’s overall dynamic characteristics and its component responses for various construction configurations, and to document how the distribution of forces within the structure changed between the various configurations. However, another fundamental objective was to establish relationships between ground motion severity, deflections, damage, and to provide data for defining realistic performance objectives. The data collected from the tests can now be used in the development of analytic models for complete buildings and the prediction of damage states and failure modes. The shake table test results also provide a basis for calibration of the CUREe-Caltech Woodframe Project’s other individual component and full-scale building test results and their integration into analytic models.
Chapter 4  
Description of Shake Table Tests

This chapter describes the four different types of shake table tests performed on the test structure: quasi-static tests, frequency evaluation tests, damping evaluation tests, and seismic tests. A description of the data biasing and filtering is provided for each of the four types of tests. In addition, a detailed description of the configuration of the test structure for each phase is provided expanding on the descriptions provided in Table 3.1. Changes in structural components such as blocking, holdowns, nailing are described.
4.1 Testing Sequences

4.1.1 Quasi-Static Tests

The main objective of the quasi-static tests was to characterize the in-plane stiffness of the floor diaphragm with appropriate boundary conditions. For the quasi-static tests, a steel channel was attached to the center of the floor diaphragm parallel to the direction of loading with lag screws into the lapped two-by-ten floor joists. The steel channel was attached with a steel link arm equipped with a load cell to the strong wall behind the house, as shown in Figure 4.1. With the center of the second floor diaphragm attached to the strong wall, the shake table was cycled three times in the north-south direction with a period of 120 seconds (0.0833 Hz) for a total test duration of 360 seconds. The data was recorded at a sampling rate of 2 Hz. All channels of data for the quasi-static tests were biased. In addition, all channels of data were filtered with a lowpass filter with a passband frequency of 0.02 Hz and a stopband frequency of 0.03 Hz with an attenuation of 60 db.

Figure 4.1: Quasi-Static Test Loading Arm
4.1.2 Frequency Evaluation Tests

The main objective of the frequency evaluation tests was to identify the natural frequencies and mode shapes of the test structure before and after each seismic test. The frequency evaluation tests of the complete structure involved inputting an acceleration-controlled flat random white noise with frequencies ranging from 1 Hz to 20 Hz for a total test duration of 165 seconds. The data was recorded at a sampling rate of 200 Hz. The data for the frequency tests was not biased nor filtered.

4.1.3 Damping Evaluation Tests

The damping evaluation tests were used to identify the equivalent viscous damping ratio of the test structure before and after each seismic test. For each of the damping evaluation tests, the complete structure was excited by the shake table with low amplitude sinusoidal motions at the first natural frequency of the test structure (as determined from the frequency evaluation tests). Once a steady-state resonance had occurred in the test structure, the shake table was suddenly stopped using a high stop ramp rate to produce decay in amplitude in motion. The data for the damping tests was not biased. Each damping test was filtered by a lowpass filter with a passband frequency 10% greater than the first natural frequency of the test structure. The stopband frequency was 20% greater than the first natural frequency of the test structure. Again, 60 db of attenuation was used with this filter. The passband filter frequency for each test can be found in the header of each damping test data file contained on an accompanying CD-ROM.
4.1.4 Seismic Tests

The main objective of the seismic tests was to determine the performance of the test structure under several levels of seismic shaking intensity. Two different types of ground motions were selected for the seismic testing: ordinary and near-field ground motions. The ground motions were selected under Task 1.3.2 of the CUREe-Caltech Woodframe Project. It was assumed that the ordinary ground motion represented a hazard level of 10% probability of exceedance in 50 years or a return period of 475 years. The 1994 Northridge Earthquake ground motion recorded at Canoga Park with an amplitude-scaling factor of 1.20 was selected as the ordinary ground motion because the acceleration response spectrum for this ground motion matched the LA NEHRP design spectrum for 10% probability of exceedance in 50 years. The acceleration time-history of the unscaled Canoga Park ground motion is shown in Figure 4.2. In addition, it was assumed that the near-field ground motion represented a hazard level of 2% probability of exceedance in 50 years or a return period of 2475 years. The 1994 Northridge Earthquake ground motion recorded at

![Acceleration Time-History of Unscaled Canoga Park Ground Motion](image)

Figure 4.2: Acceleration Time-History of Unscaled Canoga Park Ground Motion
Rinaldi was selected as the near-field ground motion. The acceleration time-history of the Rinaldi ground motion is shown in Figure 4.3. Figure 4.4 shows the acceleration response spectra for both of the unscaled ground motions for 5% damping.

![Figure 4.3: Acceleration Time-History of Rinaldi Ground Motion](image)

In addition to the 10%/50 year and 2%/50 year hazard levels, the Canoga Park ground motion was scaled to produce hazard levels of 99.99%/50 years, 50%/50 years, and 20%/50 years to test the structure under more frequently occurring ground motions. The amplitude scaling factors and the resulting peak ground accelerations are shown in Table 4.1 for all levels of seismic intensity. Up to five levels of seismic tests were performed on the test structure during each phase of dynamic testing. The input 1994 Northridge Earthquake at Canoga Park. The input ground motion for seismic test level 5 represents the unscaled 1994 Northridge Earthquake at Rinaldi. Seismic test level 4 represents an ordinary ground motion with a hazard level of 10% probability of exceedance in 50 years. Seismic test level 5 represents a near-field ground motion with a hazard level of 2% probability of exceedance in 50 years. In addition to these seismic test level ground motions, one seismic test was repeated once the inter-story
Figure 4.4: Acceleration Response Spectra of Ground motions for 5% Damping

Table 4.1: Ground Motions for Seismic Tests

<table>
<thead>
<tr>
<th>Seismic Test Level</th>
<th>Ground Motion</th>
<th>Hazard Level</th>
<th>Amplitude Scaling Factor</th>
<th>PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1994 Northridge Canoga Park</td>
<td>99.99%/50 years</td>
<td>0.12</td>
<td>0.05</td>
</tr>
<tr>
<td>2</td>
<td>1995 Northridge Canoga Park</td>
<td>50%/50 years</td>
<td>0.53</td>
<td>0.22</td>
</tr>
<tr>
<td>3</td>
<td>1996 Northridge Canoga Park</td>
<td>20%/50 years</td>
<td>0.86</td>
<td>0.36</td>
</tr>
<tr>
<td>4</td>
<td>1997 Northridge Canoga Park</td>
<td>10%/50 years</td>
<td>1.20</td>
<td>0.50</td>
</tr>
<tr>
<td>5</td>
<td>1998 Northridge Rinaldi</td>
<td>2%/50 years</td>
<td>1.00</td>
<td>0.89</td>
</tr>
</tbody>
</table>

Wall drift reached 0.5% to 1%. The experimental data for all channels of data for the seismic tests was biased. The data for the seismic tests was filtered with a lowpass
filter using a passband frequency of 12.5 Hz and a stopband frequency of 13.5 Hz with an attenuation of 60 db.

4.2 Description of Test Phases 1 – 4

The specific objective of Phases 1 – 4 was to determine the variation of the in-plane stiffness of the floor diaphragm with varying nailing, blocking, and gluing of the diaphragm through quasi-static tests. The test structure was the first story of the two-story house, as shown in Figure 4.5 and Figure 4.6. The east and west walls of the test structure were fully sheathed with no openings. The north and south walls of the test structure were framed with the window openings as specified in the construction drawings, but were not sheathed. A35 clips were installed at the second floor level as specified in the construction drawings on all four exterior walls. CS16 inter-story holdown straps were not installed for these phases of testing. HTT22 holdowns were installed only at the ends of the east and west shearwalls. HTT22 holdowns were installed in the north and south walls as specified per plans. Anchor bolts were installed in all walls as specified in the construction drawings.

During these first four phases of testing, the nailing, blocking, and gluing of the second floor diaphragm was varied, as shown in Table 4.2, to determine the influence of these parameters on the stiffness of the diaphragm. For the tests with subfloor adhesive, PL400 Subfloor Adhesive was installed on all second floor joists and rim joists. Two-by-ten and three-by-four blocking was installed at all unsupported plywood panel edges during tests with diaphragm blocking, as shown in Figure 4.7 and Figure 4.8, respectively. The unsupported plywood edges were screwed into the blocking using the design edge nail spacing for the diaphragm. One hundred percent
Figure 4.5: Phases 1 – 4 Quasi-Static Test Structure (General View)

Figure 4.6: Phases 1 – 4 Quasi-Static Test Structure (View of Floor Diaphragm)
nailing corresponds to the design 6” edge nailing and 10” field nailing for the plywood diaphragm. Fifty percent nailing corresponds to a 12” edge nailing and a 20” field nailing for the plywood diaphragm. Only quasi-static tests were performed on the test structure during Phases 1 – 4. Table 13.1 of Appendix B shows a list of the quasi-static tests with the test designations performed during Phases 1 – 4.

Table 4.2: Properties of Floor Diaphragm for Phases 1 – 4 Quasi-Static Tests

<table>
<thead>
<tr>
<th>Phase</th>
<th>Subfloor Adhesive</th>
<th>Blocking</th>
<th>Nailing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>None</td>
<td>None</td>
<td>50%</td>
</tr>
<tr>
<td>2</td>
<td>None</td>
<td>None</td>
<td>100%</td>
</tr>
<tr>
<td>3a</td>
<td>None</td>
<td>2 x 10</td>
<td>100%</td>
</tr>
<tr>
<td>3b</td>
<td>None</td>
<td>3 x 4</td>
<td>100%</td>
</tr>
<tr>
<td>4a</td>
<td>PL 400</td>
<td>None</td>
<td>50%</td>
</tr>
<tr>
<td>4b</td>
<td>PL 400</td>
<td>3 x 4</td>
<td>100%</td>
</tr>
<tr>
<td>4c</td>
<td>PL 400</td>
<td>2 x 10</td>
<td>100%</td>
</tr>
<tr>
<td>4d</td>
<td>PL 400</td>
<td>None</td>
<td>100%</td>
</tr>
</tbody>
</table>

Figure 4.7: Two-by-Ten Blocking of Floor Diaphragm
4.3 Description of Test Phase 5

The main objective of test Phase 5 was to characterize the natural frequency, damping, and seismic performance of a fully sheathed test structure with no openings. No structural repairs were made on the structure from Phases 1 – 4. The second story of the test structure was constructed for Phase 5 testing. The first and second stories of the east and west walls were fully sheathed with no openings. The north and south walls were framed with openings and sheathed, as shown in Figure 4.9 and Figure 4.10. The roof of the test structure was sheathed and covered with clay tile. A35 shear transfer clips were install on the east and west walls as specified in the plans. HTT22 holdowns were only installed at the ends of the east and west shearwalls. HTT22 holdowns were installed in the north and south walls as specified per plans. CS16 inter-story holdown straps were only installed at the ends of the east and west shearwalls. CS16 inter-story holdown straps were install on the north and south walls as specified in the plans. The interior walls of the first and second story were framed...
as specified in the plans. The second floor diaphragm was glued with PL400 Subfloor Adhesive, nailed with 100% of the specified nailing, and diaphragm blocking was removed.

Figure 4.9: Phase 5 Test Structure (View of North Wall)

Figure 4.10: Phase 5 Test Structure (View of North & East Walls)
Table 13.2 of Appendix B shows a list of the quasi-static, frequency, damping, and seismic tests with the test designations that were performed during Phase 5.

4.4 Description of Test Phase 6

In order to evaluate the natural frequency, damping, and seismic performance of a symmetric structure, window and pedestrian door openings were installed in the east and west walls for Phase 6. The OSB sheathing on the east and west walls was replaced following the testing of Phase 5. The remainder of the structure remained the same as that of Phase 5, as shown in Figure 4.11 and Figure 4.12. HTT22 holdowns were installed at the ends of the east and west shearwalls as well as adjacent to the three-foot wide door openings. CS16 inter-story holdown straps were installed at the ends of the east and west shearwalls and above the door openings. Table 13.3 of

Figure 4.11: Phase 6 Test Structure (View of North & East Walls)
Appendix B shows a list of the quasi-static, frequency, damping, and seismic tests with the test designations that were performed during test Phase 6.

![Figure 4.12: Phase 6 Test Structure (View of East Wall)](image)

### 4.5 Description of Test Phase 6A

The main objective of test Phase 6A was to determine the influence of the “waste wall” sheathing above and below the second story windows on the east and west walls. No structural repairs following Phase 6 were made prior to Phase 6A testing. In an effort to determine the influence of the “waste wall” sheathing, the sheathing above and below the second story windows on the east and west walls was removed for Phase 6A testing, as shown in Figure 4.13 and Figure 4.14. The remainder of the structure remained the same as that of Phase 6. Table 13.4 of Appendix B shows a list of the frequency, damping, and seismic tests with the test designations that were performed during Phase 6A.
4.6 Description of Test Phase 7

The main objective of test Phase 7 was to determine the difference in performance of the test structure using the perforated shearwall design method (Breyer et al., 1999). The perforated shearwall design method considers an entire wall to act as
a single unit regardless of the openings in the wall. As a result, only holdowns are required at the end of the wall, and not at the end of each shearwall segment as is required in typical shearwall design. The structure was modified using the perforated shearwall design method on the east and west walls of the test structure. Only the “wastewall” sheathing that was removed during Phase 6A was replaced for Phase 7. The configuration of the structure remained the same as that of Phase 6, as shown in Figure 4.15 and Figure 4.16. For the perforated shearwall design, the nuts for the HTT22 holdowns adjacent to the door openings on the east and west wall were removed, as shown in Figure 4.17. In addition, the CS16 inter-story holdown straps at the ends of the second story east and west wall windows were removed. However, the CS16 inter-story straps at the ends of the east and west wall remained in place. The nailing of the second story sill plate at the shearwall elements of the east and west walls was increased to resist overturning as shown in Figure 4.18. The sill nailing was
Figure 4.16: Phase 7 Test Structure (View of East Wall)

Figure 4.17: Holdown Nut Removed Adjacent to Door Openings for Phase 7
increased to 6-16d sinkers per stud bay at the full-height shearwall segments. Table 13.5 of Appendix B shows a list of the quasi-static, frequency, damping, and seismic tests with the test designations that were performed during Phase 7.

4.7 Description of Test Phase 7A

The main objective of test Phase 7A was to determine the influence of shear transfer anchor bolts near the pedestrian door openings with the perforated shearwall design method. The structure of Phase 7 was modified by installing a shear transfer anchor bolt adjacent to the east and west door openings, as shown in Figure 4.19. No structural repairs were made for Phase 7A. To maintain the same number of shear transfer anchor bolts as in Phase 7, one shear transfer anchor bolt nut was removed in each first story shearwall element, as shown in Figure 4.20. The remainder of the structure remained the same as that of Phase 7. Table 13.6 of Appendix B shows a list
of the frequency, damping, and seismic tests with the test designations that were performed during Phase 7A.

Figure 4.19: Shear Transfer Anchor Bolt Adjacent to Door Openings for Phase 7A

Figure 4.20: Anchor Bolting Typical on East & West Shearwall Elements
4.8 Description of Test Phase 8

The specific objective of test Phase 8 was to determine the frequency, damping, and seismic performance of a structure built using “conventional construction” (non-engineered). For test Phase 8, the test structure was modified to represent a “conventional construction”. Only the first story OSB sheathing on the east and west walls was replaced for Phase 8. All HTT22 holdowns on the east and west walls were removed and replaced with shear transfer anchor bolts, as shown in Figure 4.21. In addition, intermediate shear transfer anchor bolts were removed from the first story east and west shearwalls leaving only one anchor bolt at the end of each shearwall element, as shown in Figure 4.22. All CS16 inter-story holdown straps were removed from the east and west walls. In addition, all A35 shear transfer clips were removed from the east and west walls and replaced with 2-8db nails per block into the top plate of the first story east and west shearwalls, as shown in Figure 4.23. The nailing for the second story sill plate of the east and west shearwalls was increased to

Figure 4.21: Removal of Holdowns on East and West Walls for Phase 8
Figure 4.22: Removal of Intermediate Shear Transfer Anchor Bolts for Phase 8

Figure 4.23: Toenailing Through Blocking into Wall Top Plates for Phase 8

3-16ds per stud bay, as shown in Figure 4.24. The remainder of the structure remained the same as the test structure in Phase 7. Table 13.7 of Appendix B shows a list of the
quasi-static, frequency, damping, and seismic tests with the test designations that were performed during Phase 8.

4.9 Description of Test Phase 9

The main objective of test Phase 9 was to quantify the influence of a large opening in the first story east wall on the performance and torsional behavior of the structure. The test structure was returned to the configuration of Phase 6 with the exception of a garage door opening introduced in the first story east wall, as shown in Figure 4.25. The wall studs and the OSB sheathing on the east and west walls were replaced for Phase 9. All HTT22 holdowns, all shear transfer anchor bolts, all CS16 inter-story holdown straps, and all A35 shear transfer clips were reinstalled according to the plans. The nailing of the shearwall elements adjacent to the garage door opening was increased to 3” on center. Table 13.8 of Appendix B shows a list of the quasi-
static, frequency, damping, and seismic tests with the test designations that were performed during Phase 9.

Figure 4.25: Phase 9 Test Structure (View of North & East Walls)

4.10 Description of Test Phase 10

The main objective of test Phase 10 was to determine the influence of non-structural finish materials (exterior stucco and interior gypsum wallboard) on the performance of the structure. The OSB sheathing on the east and west walls was replaced for Phase 10. The exterior stucco was installed in three coats producing a total thickness of 7/8-inches, as shown in Figure 4.26. Galvanized chicken wire was fastened over tarpaper to the OSB sheathing with staples as shown in Figure 4.27 for attachment of the stucco. One-half inch interior gypsum wallboard was installed on all interior wall and ceiling surfaces and was tapped, mudded, and painted, as shown in Figure 4.28. The gypsum wallboard panels were oriented horizontally on the walls and fastened with 1 ¼” drywall screws at 16” on center. The panels were fastened with the same screws at 12” on center to the ceiling. The gypsum wallboard on the south half
of the first story walls was installed using floating edge construction in which the top row of screws was held 12 inches down from the top plate of the wall, as shown in Figure 4.29. In addition, windows and a pedestrian door were installed in the test structure. The framing, sheathing, and connection hardware remained the same as the

Figure 4.26: Phase 10 Test Structure (View of North & East Walls)

Figure 4.27: Chicken Wire Attachment to House for Stucco
test structure of Phase 9. Table 13.9 of Appendix B shows a list of the quasi-static, frequency, damping, and seismic tests with the test designations that were performed during Phase 10.
4.11 Supplemental Weight Distribution

As a result of changes in finish materials during the phases of dynamic testing, supplemental weights were installed in the test structure to maintain a constant seismic weight of 24.6 kips during all of the dynamic tests. The mass of the structure for Phase 10 was used as the reference mass because interior gypsum wallboard and exterior cement stucco were installed for this phase. Concrete paver blocks with dimensions of 16” x 24” x 2 ¼” weighing 72 pounds each were used as supplemental weights on the second story walls and the second floor diaphragm. The supplemental weight blocks were installed in stacks throughout the second story walls and second floor diaphragm as shown in Figure 13.1 of Appendix B. Concrete paver blocks with dimensions of 12” x 12” x 2” weighing 21 pounds each were used as supplemental weights mounted in the roof trusses as shown in Figure 13.2 of Appendix B.

The supplemental weight blocks were installed on the walls, floor, and roof in such a way to prevent any additional stiffness to the walls, floor, or roof, respectively. The supplemental weight blocks were installed on the walls by first mounting a 3 x 4 wood block with a 5/8” diameter all-thread protruding from the wood block to the wall studs. A rubber spacer washer was installed on the all-thread to prevent friction between the supplemental weight blocks and the wood 3 x 4 wood block. The supplemental weight blocks were then hung on the all-thread from a hole drilled in the supplemental weight block as shown in Figure 4.30. A nut with a bearing plate washer was used to secure the blocks to the wall.
For the supplemental weight blocks on the second floor, the concrete paver blocks were stacked up on the floor with rubber spacer washers installed between the bottom block and the plywood subfloor as shown in Figure 4.31. A threaded rod was installed through drilled holes in the concrete blocks and through a hole in the
plywood subfloor. Each stack of concrete blocks was then secured to the floor with a nut and bearing plate washer on each end of the threaded rod.

The supplemental weight blocks were installed in the roof trusses by first installing a 5/8” diameter threaded rod through a drilled hole in the bottom chord or center vertical member of a roof truss. A smaller concrete paver block was hung from the threaded rod on each side of the truss member through a drilled hole in the paver block as shown in Figure 4.32. Similarly, a rubber washer was installed between the supplemental weight blocks and the wood truss member. Each pair of concrete blocks was secured to the truss with a nut and bearing plate washer on each end of the all-thread.

Figure 4.32: Supplemental Weight Blocks in the Roof Trusses

The supplemental weight blocks installed on the top half of the second story walls for Phase 5 - 9 represented the weight of the exterior stucco and interior gypsum wallboard that was present on the top of the second story walls that was present in
Phase 10. Figure 13.3 of Appendix B shows the exact locations of the supplemental weight blocks on the east and west second story walls for Phase 5. Figure 13.4 of Appendix B shows the exact locations of the supplemental weight blocks on the east and west second story walls for Phases 6 – 9. Figure 13.5 of Appendix B shows the exact locations of the supplemental weight blocks on the north and south second story walls for Phases 5 – 9. The supplemental weight blocks installed on the second floor diaphragm for Phases 5 – 9 represent the weight of the gypsum wallboard installed on the first story ceiling, on the top half of the first story interior bearing wall, and on the second story interior partition walls and the weight of the interior gypsum wallboard and exterior stucco on the bottom half of the exterior second story walls and the top half of the exterior first story walls that was present in Phase 10. Figure 13.6 of Appendix B shows the exact locations of the supplemental weight blocks on the second floor diaphragm for Phases 5 – 9. The supplemental weight blocks installed in the roof trusses for Phase 5 – 9 represent the weight of the second story gypsum wallboard ceiling that was present in Phase 10. In addition, 8 weight blocks were installed on the center vertical member of the roof trusses to represent the weight of miscellaneous materials (insulation, plumbing, electrical, etc.) that were not present. Figure 13.7 of Appendix B shows the exact locations of the supplemental weight blocks in the roof trusses for Phases 5 – 9. Figure 13.8 of Appendix B shows the exact locations of the supplemental weight blocks in the roof trusses for Phase 10.
Chapter 5  Description of Instrumentation for Shake Table Tests

The test structure was instrumented with nearly three-hundred digital instruments to measure forces, displacements, and accelerations in the structure during the shake table tests. Due to changes in the configuration of the test structure with each phase, the number and locations of the instruments changed with each phase. Table 13.10 of Appendix C shows the channel name and the corresponding measurement for each instrument used in the test structure. In addition, this table shows the engineering units of the measured data as well as the sign convention used with the instruments. Note that this is a comprehensive list of all of the instruments used in the test structure. For most phases of testing, not all instruments were active.
The instruments used in a particular test can be found in the graphical instrumentation plans for each phase in Appendix C. Descriptions and general locations of each type of instrument follow.

5.1 Types of Instrumentation for Shake Table Tests

5.1.1 Force Measurements

For all of the shake table tests, tension forces in the shear transfer anchor bolts and holdown anchor bolts were measured with load cells as shown in Figure 5.1. The load cells were manufactured using a length of steel pipe that had an inside diameter slightly larger than the anchor bolt. Four strain gages were attached to the steel pipe (2 parallel to longitudinal direction and 2 perpendicular to longitudinal direction) and

Figure 5.1: Typical Anchor Bolt Load Cell
were wired with a full bridge. An 11-kip load cell was also used to measure the axial force in the quasi-static loading arm for the quasi-static tests as was shown previously in Figure 4.1.

5.1.2 Displacement Measurements

Absolute displacements of the south elevation and roof of the test structure were measured with string potentiometers attached to the strong wall behind the house as shown in Figure 5.2. Shear or racking (diagonal) deformations of all of the shearwall elements were measured with linear potentiometers mounted at one end of telescoping aluminum tubes. In addition, vertical deformations of the ends of shearwall elements on the east and west elevations were measured with the same type

Figure 5.2: String Potentiometers at South Side of Test Structure
of linear potentiometers mounted at one end of telescoping aluminum tubes. Typical vertical and racking (shear) deformation instruments on the shearwalls are shown in Figure 5.3. The shear or racking (diagonal) deformation of the floor diaphragm was measured with five sets (seven sets for Phases 1 – 4) of linear potentiometers mounted at one end of telescoping aluminum tubes as shown in Figure 5.4. A similar instrumentation arrangement was used to measure the flexural deformation of the second floor diaphragm at the north and south edges of the diaphragm. Linear potentiometers were used to measure the slippage of the sill plate relative to the steel base on the first floor and relative to the floor diaphragm on the second floor as shown in Figure 5.5. The uplift of the sill plates at all shearwall element ends were measured using linear potentiometers relative to the steel base on the first floor and relative to the second floor rim joists on the second floor. In addition, the uplift of the holdown studs at shearwall element ends were measured.
with linear potentiometers relative to the steel base on the first floor and relative to the second floor rim joists on the second floor. Figure 5.6 shows typical sill plate and holdown stud uplift measurement instruments at the first story.
5.1.3 Acceleration Measurements

For the dynamic shake table tests, 61 accelerometers were used to measure accelerations throughout the structure. Accelerometers were placed on the south wall elevation and roof of the test structure at the same locations where absolute horizontal displacements were measured with string potentiometers. In addition, several accelerometers were placed on the east wall elevation of the test structure to capture the torsional response. An array of 11 evenly spaced accelerometers was installed across the floor diaphragm as shown in Figure 5.7. These accelerometers were used to determine the distribution of inertia forces in the floor diaphragm. Similarly, an array of 9 evenly spaced accelerometers was installed across the roof diaphragm midway between the ridge and the eaves of the roof on the south side of the test structure as
shown in Figure 5.8. Again, these accelerometers were used to determine the distribution of inertia forces in the roof diaphragm.

Figure 5.7: Array of Accelerometers on Floor Diaphragm

Figure 5.8: Array of Accelerometers on Roof Diaphragm
5.2 Instrumentation for Test Phases 1 – 4

Since only quasi-static tests were performed during Phases 1 – 4, acceleration measurements were not performed. During these tests, the axial load in the quasi-static loading arm was monitored with an 11-kip load cell (Channel U1). Figure 13.9, Figure 13.10, and Figure 13.11 of Appendix C show instrumentation plans of the test structure for Phases 1 – 4. Table 13.11 of Appendix C shows the initial lengths of the telescoping aluminum tubes (Channels H, L, & M) that were required for shear or racking deformation calculations.

5.3 Instrumentation for Test Phase 5

Accelerations, displacements, and forces were measured for the quasi-static and dynamic testing that was performed during Phase 5. Figure 13.12, Figure 13.13, and Figure 13.14 of Appendix C show instrumentation plans of the test structure for Phase 5. Table 13.12 of Appendix C shows the initial lengths of the telescoping aluminum tubes (Channels H, L, & M) that were required for shear or racking deformation calculations.

5.4 Instrumentation for Test Phases 6 & 6A

Accelerations, displacements, and forces were measured for the quasi-static and dynamic testing that was performed during Phases 6 & 6A. Since small door openings and window openings were installed on the east and west wall elevations for Phases 6 & 6A, changes in the instrumentation included modification of the aluminum telescoping racking deformation instrumentation and installation of addition linear potentiometers for measurement of uplift at shearwall element ends. Figure 13.15, Figure 13.16, and Figure 13.17 of Appendix C show instrumentation plans of the test
structure for Phases 6 & 6A. Table 13.13 of Appendix C shows the initial lengths of the telescoping aluminum tubes (Channels H, L, & M) that were required for shear or racking deformation calculations.

5.5 Instrumentation for Test Phase 7

Accelerations, displacements, and forces were measured for the quasi-static and dynamic testing that was performed during Phase 7. With the perforated shearwall configuration in Phase 7, the only changes from the instrumentation from Phases 6 & 6A was the removal of the holdown anchor bolt load cells adjacent to the door openings on the east and west wall (Channels F16, F17, F26, and F27). Figure 13.18, Figure 13.19, and Figure 13.20 of Appendix C show instrumentation plans of the test structure for Phase 7. Table 13.13 of Appendix C shows the initial lengths of the telescoping aluminum tubes (Channels H, L, & M) that were required for shear or racking deformation calculations.

5.6 Instrumentation for Test Phase 7A

Accelerations, displacements, and forces were measured for the dynamic testing that was performed during Phase 7A. The only changes from the instrumentation from Phase 7 include the reinstallation of anchor bolt load cells adjacent to the door openings (Channels F16, F17, F26, and F27) and the removal of one of the shear transfer anchor bolts in each shearwall element of the first story east and west walls (Channels F15, F18, F25, and F28). Figure 13.21, Figure 13.22, and Figure 13.23 of Appendix C show instrumentation plans of the test structure for Phase 7A. Table 13.13 of Appendix C shows the initial lengths of the telescoping aluminum tubes (Channels H, L, & M) that were required for shear or racking deformation calculations.
tubes (Channels H, L, & M) that were required for shear or racking deformation calculations.

### 5.7 Instrumentation for Test Phase 8

Accelerations, displacements, and forces were measured for the quasi-static and dynamic testing that was performed during Phase 8. The instrumentation of the structure was returned to that of Phases 6 & 6A with the exception of the removal of intermediate shear transfer anchor bolt load cells on each shearwall element of the first story east and west walls (Channels F14, F15, F18, F19, F24, F25, F28, F29). Figure 13.24, Figure 13.25, and Figure 13.26 of Appendix C show instrumentation plans of the test structure for Phase 8. Table 13.13 of Appendix C shows the initial lengths of the telescoping aluminum tubes (Channels H, L, & M) that were required for shear or racking deformation calculations.

### 5.8 Instrumentation for Test Phase 9

Accelerations, displacements, and forces were measured for the quasi-static and dynamic testing that was performed during Phase 9. The instrumentation of the structure was the same as that of Phases 6 & 6A with modifications to the aluminum telescoping racking deformation instrumentation and anchor bolt load cells for the garage door openings on the first story east wall. Figure 13.27, Figure 13.28, and Figure 13.29 of Appendix C show instrumentation plans of the test structure for Phase 9. Table 13.14 of Appendix C shows the initial lengths of the telescoping aluminum tubes (Channels H, L, & M) that were required for shear or racking deformation calculations.
5.9 Instrumentation for Test Phase 10

Accelerations, displacements, and forces were measured for the quasi-static and dynamic testing that was performed during Phase 10. The instrumentation of the structure was the same as that of Phase 9 with the exception of the removal of telescoping aluminum tubes instrumentation on the second floor diaphragm (Channels L1 – L10) due to interference with drywall on the second floor walls. Figure 13.30, Figure 13.31, and Figure 13.32 of Appendix C show instrumentation plans of the test structure for Phase 10. Table 13.15 of Appendix C shows the initial lengths of the telescoping aluminum tubes (Channels H, L, & M) that were required for shear or racking deformation calculations.
Chapter 6  Determination of Material Properties

6.1  Moisture Content and Specific Gravity of Wood Materials

In an effort to limit variation of moisture content in the framing lumber and sheathing materials over the several months of testing, “dry” lumber with a maximum moisture content of 19% was used in the test structure. By using “dry” framing lumber, the shrinkage of the lumber and resulting effects on nailed connections would be minimized. Over the duration of the construction and testing of the test structure, the moisture content of the framing lumber and sheathing materials were tested according to ASTM D4442 using “Method A—Primary Oven-Drying Method” (American Society for Testing and Materials, 1997).
Wood samples were stored in close proximity to the test structure to ensure the moisture content of the samples matched that of the framing lumber and sheathing materials of the test structure. Samples with approximate dimensions of 3.5” x 5” x 1.5” were used for the measurement of the moisture content for 2 x 4 studs and top plates, 2 x 4 pressure treated sill plates, and 2 x 10 floor joists. Samples with approximate dimensions of 8” x 8” were used for the measurement of the moisture content for the 3/8” OSB wall sheathing, ¾” plywood subfloor, and ½” plywood roof sheathing. Samples were initially weighed using a balance and then placed in a forced-convection oven at 103 degrees Celsius for at least 12 hours to ensure all of the moisture from samples had been removed. The samples were weighed again with a balance after removing the samples from the oven. The moisture content for each sample was determined as follows:

\[
MC = \frac{\text{Original Mass} - \text{Oven-Dry Mass}}{\text{Oven-Dry Mass}} \times 100\% \quad (6.1)
\]

The moisture content tests were performed every one to two weeks for the first four months of the testing of the house. Once it became evident that the moisture content for samples was well below the 19% limit, moisture content tests were performed monthly. Figure 6.1 shows the variation of the moisture content for the framing lumber and sheathing materials. Variability in the moisture content of a particular type of sample can be attributed to change in climatic conditions and the selection of a “random sample” of materials.

The specific gravity of the framing lumber and sheathing materials was measured for use in computing material weights and comparison to standard values of
specific gravity for the wall sheathing materials established under the CUREe-Caltech
Woodframe Project. Table 6.1 shows the specific gravity and density of six types of
wood materials that were used in the test structure.

![Figure 6.1: Variation of Moisture Content of Wood Materials During Testing](image)

Table 6.1: Specific Gravity of Test Structure Building Materials

<table>
<thead>
<tr>
<th>Building Material</th>
<th>Specific Gravity</th>
<th>Density (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 x 4 Stud and Top Plate</td>
<td>0.55</td>
<td>34.5</td>
</tr>
<tr>
<td>2 x 4 Pressure Treated Sill Plate</td>
<td>0.46</td>
<td>28.4</td>
</tr>
<tr>
<td>2 x 10 Floor Joist</td>
<td>0.56</td>
<td>35.1</td>
</tr>
<tr>
<td>3/8&quot; OSB Wall Sheathing</td>
<td>0.62</td>
<td>38.9</td>
</tr>
<tr>
<td>3/4&quot; Plywood Subfloor</td>
<td>0.58</td>
<td>36.1</td>
</tr>
<tr>
<td>1/2&quot; Plywood Roof Sheathing</td>
<td>0.57</td>
<td>35.6</td>
</tr>
</tbody>
</table>

6.2 Hysteretic Properties of Sheathing to Framing Lumber Connections

Connection tests of nailed sheathing to framing assemblies were performed to
determine the hysteretic properties of the connections for use in analytical modeling.
Four types of connection tests were performed using the sheathing and framing materials from the house test structure. In two of the connection test configurations, the specimens were loaded in such a way to produce nail deformation parallel to the grain of the framing lumber. For the other two connection test configurations, the specimens were loaded in such a way to produce nail deformation perpendicular to the grain of the framing lumber. Connection specimens with 3/8” thick Oriented Strand Board (OSB) sheathing and ¾” thick subfloor plywood sheathing were tested. Table 6.2 shows the four configurations of the connection tests.

Displacement-controlled monotonic tests were performed for all connection test configurations. In addition, displacement-controlled cyclic tests were performed for the OSB connection test specimens. For each configuration, a minimum of three monotonic pull tests were performed on test samples until failure. From these monotonic tests, a particular displacement, \( \Delta \), was obtained for use in the subsequent cyclic connection tests for the OSB specimens that followed the cyclic loading protocol developed under Task 1.3.2 of the CUREe-Caltech Woodframe Project (Krawinkler et al., 2000). The displacement, \( \Delta \), was defined as 60% of the

<table>
<thead>
<tr>
<th>Loading Direction to Relative Grain</th>
<th>Sheathing</th>
<th>Nailing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parallel</td>
<td>3/8” OSB</td>
<td>8d box gun nails</td>
</tr>
<tr>
<td>Parallel</td>
<td>3/4” Plywood</td>
<td>10d (0.131” diameter) gun nails</td>
</tr>
<tr>
<td>Perpendicular</td>
<td>3/8” OSB</td>
<td>8d box gun nails</td>
</tr>
<tr>
<td>Perpendicular</td>
<td>3/4” Plywood</td>
<td>10d (0.131” diameter) gun nails</td>
</tr>
</tbody>
</table>
displacement occurring at 80% of the maximum load in the test specimen. This
displacement was then used with the CUREe-Caltech Woodframe Project Simplified
Cyclic Loading Protocol as shown in Figure 6.2 to develop the cyclic loading protocol
for each test specimen. All specimens were tested using a loading rate of 0.3 inches
per minute.

Figure 6.2: CUREe-Caltech Woodframe Project Simplified Cyclic Loading Protocol
(After Krawinkler et al., 2000)

The parallel to grain loading specimens were constructed using two 10” two-
by-four pieces of framing lumber joined together with a 5” x 10” piece of sheathing on
each side as shown in Figure 6.3. The top half of each piece of sheathing was nailed to
the top piece of lumber with two gun nails. The bottom half of each piece of sheathing
was fastened to the bottom piece of lumber with two gun nails and one screw along
Figure 6.3: Parallel to Grain Connection Test Configuration

with epoxy to produce a fixed condition. With this fixed condition at the bottom piece of lumber, only the top four nails (two each side) underwent deformation.

The parallel to grain loading specimens were installed in a 110-kip MTS 810 Material Test System machine using custom steel brackets with two steel pins through each piece of framing lumber as shown in Figure 6.4. The machine applied an axial
load to each specimen producing parallel to grain deformation in the framing members.

The perpendicular to grain loading specimens were constructed using one 10” two-by-four piece of framing lumber with a 5” x 10” piece of sheathing edge nailed on each side as shown in Figure 6.5. Each piece of sheathing was nailed at the edge to the two-by-four piece of lumber with three gun nails (6 total). The perpendicular to grain loading specimens were installed in a 110-kip MTS 810 Material Test System machine using custom steel brackets at the top with two steel pins through each piece of sheathing as shown in Figure 6.6. A flat bar placed across the surface of the two-by-
four framing lumber was bolted to another bracket in the MTS machine creating a
distributed load on the lumber when loaded. The machine applied an axial load to the
brackets producing perpendicular to grain deformation in the framing-to-sheathing
connections.

The connection test specimens were instrumented with a linear potentiometer
to measure the displacement of the sheathing relative to the framing lumber. For the
parallel to grain loading test specimens, a linear potentiometer was installed to
measure the displacement between the two pieces of framing lumber. A linear
potentiometer was installed on the perpendicular to grain loading test specimens to
measure the displacement of the sheathing relative to the MTS 810 Material Test
System machine (or equivalently to the framing lumber since it was held fixed in

Figure 6.5: Perpendicular to Grain Connection Test Configuration
Appendix D shows plots of load versus deformation for the monotonic tests and load-deformation hysteresis loops for the cyclic tests of the sheathing to framing connection specimens. Table 6.3 shows the variability in the peak load and the corresponding deformation for each of the connection tests.
Table 6.3: Peak Load and Deformation at Peak Load for Connection Tests

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Sheathing</th>
<th>Loading</th>
<th>Specimen</th>
<th>Peak Load (lbs)</th>
<th>Deformation at Peak Load (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parallel</td>
<td>OSB</td>
<td>Monotonic</td>
<td>Specimen 1</td>
<td>222</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen 2</td>
<td>165</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen 3</td>
<td>95</td>
<td>0.29</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen 4</td>
<td>308</td>
<td>0.70</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cyclic</td>
<td>Specimen 1</td>
<td>267</td>
<td>0.45</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen 2</td>
<td>230</td>
<td>0.24</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen 3</td>
<td>290</td>
<td>0.45</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen 4</td>
<td>173</td>
<td>0.19</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen 5</td>
<td>252</td>
<td>0.44</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen 6</td>
<td>239</td>
<td>0.44</td>
</tr>
<tr>
<td>Perpendicular</td>
<td>OSB</td>
<td>Monotonic</td>
<td>Specimen 1</td>
<td>447</td>
<td>0.47</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen 2</td>
<td>374</td>
<td>0.66</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen 3</td>
<td>414</td>
<td>0.38</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cyclic</td>
<td>Specimen 1</td>
<td>212</td>
<td>0.32</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen 2</td>
<td>210</td>
<td>0.32</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen 3</td>
<td>240</td>
<td>0.45</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen 4</td>
<td>239</td>
<td>0.46</td>
</tr>
<tr>
<td>Parallel</td>
<td>Plywood</td>
<td>Monotonic</td>
<td>Specimen 1</td>
<td>321</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen 2</td>
<td>262</td>
<td>0.19</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen 3</td>
<td>318</td>
<td>0.28</td>
</tr>
<tr>
<td>Perpendicular</td>
<td>Plywood</td>
<td>Monotonic</td>
<td>Specimen 1</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen 2</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen 3</td>
<td>*</td>
<td>*</td>
</tr>
</tbody>
</table>

* Data unavailable at time of writing

6.3 Compressive Strength of Exterior Stucco

Samples of the exterior cement stucco were taken during the application of the stucco for compressive strength testing. The test sample cylinders were 4 inches tall with a diameter of 1.5 inches. Three stucco samples were taken during each of the three coats of exterior stucco: scratch coat, brown coat, and color coat. Each of the samples was tested in a compression test machine to determine the compressive strength of the stucco. Table 6.4 shows the average compressive strength for each layer of exterior stucco.
Table 6.4: Compressive Strength of Exterior Stucco

<table>
<thead>
<tr>
<th>Stucco Material</th>
<th>$f'_c$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scratch Coat</td>
<td>0.85 +/- 0.04</td>
</tr>
<tr>
<td>Brown Coat</td>
<td>1.07 +/- 0.21</td>
</tr>
<tr>
<td>Color Coat</td>
<td>0.35 +/- 0.09</td>
</tr>
</tbody>
</table>
Chapter 7  Results of Quasi-Static Tests

In this chapter, the results of the quasi-static tests are used to determine the influence of diaphragm nailing, diaphragm blocking, diaphragm gluing, and the presence of the second story on the in-plane stiffness of the floor diaphragm. The equivalent viscous damping ratio is computed for each test using the global diaphragm hysteretic behavior. The flexibility of the diaphragm for each test is classified according to the 1997 edition of the Uniform Building Code (ICBO, 1997). In addition, a model to describe the observed phase lag between shear and global deformations in the floor diaphragm is presented. Recommendations concerning building codes and current design practices are discussed using the results of these quasi-static tests.
7.1 Diaphragm Analysis

7.1.1 Global Diaphragm Deformation

In order to compare the effects of diaphragm nailing, gluing, blocking, and structural configuration, the overall global stiffness of the second floor diaphragm was first determined. As the reaction arm applied a concentrated load to the center of the second floor diaphragm as the shake table was cycled, the center of the diaphragm deflected relative to the diaphragm ends (located at the east and west shearwalls) as shown in Figure 7.1. In addition, the east and west shearwalls deflected with the applied load. The global diaphragm deflection, $\Delta_{\text{global}}$, is defined as the total deflection at the center of the diaphragm relative to the shake table, $\Delta_{\text{total}}$, minus the deflection of the shearwalls relative to the shake table, $\Delta_{\text{shearwall}}$.

![Figure 7.1: Deformation of Floor Diaphragm](image)
Global load-deflection hysteresis loops for the global diaphragm behavior were developed for each phase of testing. Figure 7.2 shows these hysteresis loops for Phase 1 testing. The experimental global diaphragm stiffness was computed using:

\[
k_{\text{global}} = \frac{|F_{\text{min}}| + |F_{\text{max}}|}{\Delta_{\text{global, min}} + \Delta_{\text{global, max}}} \tag{7.1}
\]

where \(F_{\text{min}}\) and \(F_{\text{max}}\) represent the minimum and maximum force applied to the floor diaphragm through the reaction arm, respectively, while \(\Delta_{\text{global, min}}\) and \(\Delta_{\text{global, max}}\) are the corresponding minimum and maximum global deformation.

![Figure 7.2: Global Load-Deformation Hysteresis Loops of Floor Diaphragm for Phase 1 Testing](image_url)
7.1.2 Diaphragm Shear Deformation

The global diaphragm deformation can be divided into diaphragm shear deformation and diaphragm flexural deformation. The diaphragm shear deformation was measured using a series of diagonal racking deformation instruments located on the second floor diaphragm as shown in Figure 13.11 of Appendix C. Since a concentrated load was applied to the center of the diaphragm, it is assumed that the shear deformation for each half of the floor diaphragm could be computed based on the configuration of the diagonal instrumentation and structure as shown in Figure 7.3. The shear deformation in the floor diaphragm, \( \Delta_s \), can be expressed as:

\[
\Delta_s = \gamma \cdot \frac{L}{2}
\]  

(7.2)

where \( \gamma \) is the shear strain that can be determined from the diagonal deformation measurements using:

\[
\gamma = \frac{\Delta_L \sqrt{b^2 + d^2}}{bd}
\]  

(7.3)

where \( \Delta_L \) is the average diagonal deformation from a pair of diagonal deformation instruments, \( b \) is the width of a pair of diagonal deformation instruments as shown in Figure 7.3, and \( d \) is the depth of the diaphragm.

Shear load-deformation hysteresis loops for the floor diaphragm can be constructed. Figure 7.4 shows these hysteresis loops for Phase 1 testing. The experimental diaphragm shear stiffness was computed using:

\[
k_{\text{shear}} = \frac{|V_{\text{min}}| + |V_{\text{max}}|}{|\Delta_{\text{shear, min}}| + |\Delta_{\text{shear, max}}|}
\]  

(7.4)
where $V_{\text{min}}$ and $V_{\text{max}}$ represent the minimum and maximum shear force applied to the floor diaphragm, respectively, while $\Delta_{\text{shear, min}}$ and $\Delta_{\text{shear, max}}$ are the corresponding minimum and maximum shear deformation. Note that the shear force is one-half of the total force applied to the diaphragm. Using elastic beam theory, the theoretical shear deformation of the floor diaphragm with a concentrated load applied at its center is:

$$\Delta_s = \frac{V \cdot \frac{L}{2}}{G_{A_s}}$$  \hspace{1cm} (7.5)$$

where $G_{A_s}$ is the shear stiffness of the diaphragm. Equation 7.5 can be rearranged to solve for $G_{A_s}$ in terms of $k_{\text{shear}}$ (shear force divided by shear deformation):

$$G_{A_s} = k_{\text{shear}} \cdot \frac{L}{2}$$  \hspace{1cm} (7.6)$$
This shear stiffness, $G_A$, was computed for each of the quasi-static tests.

$$\gamma = \frac{\Delta_l \sqrt{b^2 + d^2}}{bd}$$

$b = 47.8\text{in} \cdot d = 190.4\text{in}$

$\Delta_l = $ Average diagonal deformation

$$\gamma = \frac{\Delta_l \sqrt{(47.8\text{in})^2 + (190.4\text{in})^2}}{(47.8\text{in}) \cdot (190.4\text{in})}$$

$\gamma = 0.0216 \cdot \Delta_L$

$\Delta_l = \gamma \frac{\Delta_L}{0.0216} = 2.59 \cdot \Delta_L \text{ in}$

$-0.04 \text{ in}$

$-4.50 \text{ kips}$

$-0.20 -0.15 -0.10 -0.05 0.00 0.05 0.10 0.15 0.20$

Figure 7.4: Shear Load-Deformation Hysteresis Loops of Floor Diaphragm for Phase 1 Testing

### 7.1.3 Diaphragm Flexural Deformation

The flexural deformation of the floor diaphragm, $\Delta_{flexural}$, can be determined by subtracting the shear deformation from the global deformation of the diaphragm:

$$\Delta_{flexural} = \Delta_{global} - \Delta_{shear}$$  \hspace{1cm} (7.7)  

Flexural load-deformation hysteresis loops for the floor diaphragm were then constructed. Figure 7.5 shows these hysteresis loops for Phase 1 testing. The experimental diaphragm flexural stiffness, $k_{flexural}$, was computed using:
\[ k_{\text{flexural}} = \frac{|F_{\text{min}}| + |F_{\text{max}}|}{\Delta_{\text{flexural, min}} + \Delta_{\text{flexural, max}}} \]  

(7.8)

where \( F_{\text{min}} \) and \( F_{\text{max}} \) represent the minimum and maximum force applied to the floor diaphragm, respectively, while \( \Delta_{\text{flexural, min}} \) and \( \Delta_{\text{flexural, max}} \) are the corresponding minimum and maximum flexural deformation.

Using elastic beam theory, the theoretical flexural deformation of the floor diaphragm with a concentrated load applied at its center is:

\[ EI = \frac{k_{\text{flexural}} L^4}{48} = \left(194.4 \frac{k}{\text{kip}}\right) \left(\frac{249 \text{ in}}{48}\right)^3 \]

\[ EI = 56.0 \times 10^6 \text{ kip} \cdot \text{in}^2 \]
\[ \Delta_{\text{flexural}} = \frac{FL^3}{48EI} \]  

(7.9)

where \( EI \) is the flexural stiffness of the diaphragm. Equation 7.9 can be rearranged to solve for \( EI \) in terms of \( k_{\text{flexural}} \) (applied force divided by flexural deformation):

\[ EI = k_{\text{flexural}} \frac{L^3}{48} \]  

(7.10)

The flexural stiffness, \( EI \), was computed for each of the quasi-static tests.

### 7.1.4 Damping

The damping in the floor diaphragm can be estimated if the diaphragm is assumed to behave as a single-degree-of-freedom mass-spring system with a Kelvin solid viscoelastic hysteresis as shown in Figure 7.6.

![Figure 7.6: Kelvin Solid Viscoelastic Model of Floor Diaphragm](image-url)
The experimental energy dissipated per cycle, $E_d$, is the area enclosed by the hysteresis loop. Assuming the diaphragm behaves as a Kelvin solid viscoelastic element, the energy dissipated per cycle is (Filiatrault, 2000):

$$E_d = \pi \omega X_o^2 \bar{c} \quad (7.11)$$

where $X_o$ is the maximum displacement, $\omega$ is the natural frequency of the mass-spring system, and:

$$\bar{c} = 2 \xi_{eq} \omega^2 m \quad (7.12)$$

where $\xi_{eq}$ is the equivalent viscous damping ratio and $m$ is the mass of the mass-spring system. Substituting Equation 7.12 into Equation 7.11 gives:

$$E_d = 2 \pi \xi_{eq} \omega^2 m X_o^2 \quad (7.13)$$

The secant stiffness of the diaphragm at $X_o$ can be expressed as:

$$k_o = \omega^2 m \quad (7.14)$$

Substituting Equation 7.14 into Equation 7.13 gives:

$$E_d = 2 \pi \xi_{eq} \omega^2 m X_o^2 \quad (7.13)$$

The area of triangle OAB, $A_{OAB}$, in Figure 7.6 can be expressed as:

$$A_{OAB} = \frac{X_o^2 k_o}{2} \quad (7.16)$$

Substituting Equation 7.16 into Equation 7.15 gives:

$$E_d = 4 \pi \xi_{eq} A_{OAB} \quad (7.17)$$

Solving for the equivalent viscous damping ratio give:

$$\xi_{eq} = \frac{E_d}{4 \pi A_{OAB}} \quad (7.18)$$
Since the maximum and minimum global diaphragm displacement were not necessarily equal, equivalent viscous damping ratios were computed for positive displacement (triangle OAB in Figure 7.6) and negative displacement (triangle OCD in Figure 7.6) and then averaged.

### 7.2 Variation of Global Stiffness, Shear Stiffness, Flexural Stiffness, and Damping of Diaphragm

The global diaphragm stiffness, $k_{\text{global}}$, diaphragm shear stiffness, $GA$, diaphragm flexural stiffness, $EI$, and diaphragm equivalent viscous damping were determined for each of the quasi-static tests performed on the structure. Appendix E shows plots of the global diaphragm deformation hysteresis, diaphragm shear deformation hysteresis, and diaphragm flexural deformation hysteresis and the corresponding stiffness values for each of the quasi-static tests. These graphs show significant hysteretic behavior for the nailed diaphragm floor. Table 7.1 shows a summary of the diaphragm stiffness and equivalent viscous damping for each of the quasi-static tests performed.

#### 7.2.1 Discussion on Diaphragm Stiffness Variation

Several important trends and relationships can be seen by comparing the diaphragm stiffness for each of the test phases. There was only a small increase in diaphragm stiffness between Phases 1 and 2 when the nailing was increased from 50% to 100%. Both of these tests show a low shear stiffness for diaphragms without blocking and subfloor adhesive. There was a significant increase of the shear stiffness of the diaphragm for Phases 3A and 3B with the addition of blocking at unsupported plywood panel edges. The diaphragm blocked with 3 x 4 blocking had a higher shear
Table 7.1: Summary of Diaphragm Stiffness and Damping for Quasi-Static Tests

<table>
<thead>
<tr>
<th>Test Phase</th>
<th>Structure Configuration</th>
<th>Global Stiffness, $k_{global}$ (kips/in)</th>
<th>Shear Stiffness, $GA_s$ (kips)</th>
<th>Flexural Stiffness, $EI$ (kip-in$^2$)</th>
<th>Equivalent Viscous Damping, $\zeta_{eq}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>One story, 50% nailing, no blocking, no adhesive</td>
<td>73.5</td>
<td>6,657</td>
<td>$56 \times 10^6$</td>
<td>22.5%</td>
</tr>
<tr>
<td>2</td>
<td>One story, 100% nailing, no blocking, no adhesive</td>
<td>86.0</td>
<td>7,143</td>
<td>$70 \times 10^6$</td>
<td>19.1%</td>
</tr>
<tr>
<td>3A</td>
<td>One story, 100% nailing, 2 x 10 blocking, no adhesive</td>
<td>109.3</td>
<td>19,708</td>
<td>$45 \times 10^6$</td>
<td>13.5%</td>
</tr>
<tr>
<td>3B</td>
<td>One story, 100% nailing, 3 x 4 blocking, no adhesive</td>
<td>114.6</td>
<td>32,322</td>
<td>$41 \times 10^6$</td>
<td>13.7%</td>
</tr>
<tr>
<td>4A</td>
<td>One story, 50% nailing, no blocking, PL400 adhesive</td>
<td>105.3</td>
<td>24,354</td>
<td>$40 \times 10^6$</td>
<td>13.6%</td>
</tr>
<tr>
<td>4B</td>
<td>One story, 100% nailing, 3 x 4 blocking, PL400 adhesive</td>
<td>110.1</td>
<td>46,071</td>
<td>$36 \times 10^6$</td>
<td>12.6%</td>
</tr>
<tr>
<td>4C</td>
<td>One story, 100% nailing, 2 x 10 blocking, PL400 adhesive</td>
<td>134.7</td>
<td>51,408</td>
<td>$45 \times 10^6$</td>
<td>12.5%</td>
</tr>
<tr>
<td>4D</td>
<td>One story, 100% nailing, no blocking, PL400 adhesive</td>
<td>101.7</td>
<td>31,580</td>
<td>$36 \times 10^6$</td>
<td>13.6%</td>
</tr>
<tr>
<td>5</td>
<td>Two stories, fully sheathed, 100% nailing, no blocking, PL400 adhesive</td>
<td>296.6</td>
<td>36,450</td>
<td>$157 \times 10^6$</td>
<td>5.6%</td>
</tr>
<tr>
<td>6</td>
<td>Two stories, small openings, 100% nailing, no blocking, PL400 adhesive</td>
<td>277.6</td>
<td>30,225</td>
<td>$158 \times 10^6$</td>
<td>6.0%</td>
</tr>
<tr>
<td>7</td>
<td>Two stories, perforated shearwall design, 100% nailing, no blocking, PL400 adhesive</td>
<td>238.3</td>
<td>87,992</td>
<td>$81 \times 10^6$</td>
<td>8.1%</td>
</tr>
<tr>
<td>8</td>
<td>Two stories, conventional construction, 100% nailing, no blocking, PL400 adhesive</td>
<td>217.1</td>
<td>31,283</td>
<td>$103 \times 10^6$</td>
<td>6.6%</td>
</tr>
<tr>
<td>9</td>
<td>Two stories, large opening, 100% nailing, no blocking, PL400 adhesive</td>
<td>216.6</td>
<td>40,928</td>
<td>$89 \times 10^6$</td>
<td>8.3%</td>
</tr>
<tr>
<td>10</td>
<td>Two stories, finish materials, 100% nailing, no blocking, PL400 adhesive</td>
<td>425.7</td>
<td>--</td>
<td>--</td>
<td>10.1%</td>
</tr>
</tbody>
</table>
stiffness than the diaphragm with 2 x 10 blocking. The flexural stiffness of the diaphragm is relatively constant for these tests.

There was a significant increase in the diaphragm stiffness when the subfloor adhesive was installed for Phases 4A to 4D. The shear stiffness of the diaphragm increased by a factor of four with the installation of the subfloor adhesive when comparing Phase 1 with Phase 4A for 50% nailing. This same increase in shear stiffness was evident with the installation of subfloor adhesive when comparing Phase 2 with Phase 4D for 100% nailing. Again, there was a significant increase in the shear stiffness of the blocked diaphragm with the addition of subfloor adhesive compared to the blocked diaphragms without subfloor adhesive. These results also suggest that the influence of blocking on the diaphragm stiffness is not as significant with subfloor adhesive installed as compared to the diaphragms without subfloor adhesive. In addition, the nail spacing (50% or 100%) has little influence on the diaphragm stiffness once blocking and/or subfloor adhesive is installed.

After the second story was constructed, there was not a significant change in the diaphragm shear stiffness from that of Phase 4D. However, the flexural stiffness of the diaphragm with the second story increased significantly. This increase in flexural stiffness may have been caused by the north and south walls on the second story acting as flanges to restrain the bending deformation of the diaphragm. Note that the shear stiffness for Phase 7 is significantly higher than the other tests, but this may have been the result of faulty instrumentation and not necessarily a stiffer diaphragm. The global stiffness of the diaphragm increased dramatically with the installation of finish materials on the structure during Phase 10 when compared to Phase 9.
7.2.2 Discussion on Damping Variation

In addition to trends in diaphragm stiffness between tests phases, there were several patterns in the equivalent viscous damping of the diaphragm as shown in Table 7.1. The equivalent viscous damping was the highest at 22.5% of critical for Phases 1 and 2 as a result of low shear stiffness and significant hysteretic behavior. This low shear stiffness created friction between the sheathing and framing causing the high damping. The equivalent viscous damping decreased to around 14% of critical with the installation of blocking during Phases 3A and 3B. The shear stiffness increased for these two phases causing the decrease in equivalent viscous damping. The equivalent viscous damping in the diaphragm remained roughly the same around 13% with the installation of subfloor adhesive for Phase 4A to Phase 4D. For Phase 5 to 9, the equivalent viscous damping in the diaphragm decreased substantially (5% to 8% of critical) with the construction of the second story. This range of equivalent viscous damping in the diaphragm matches well with that found during the dynamic tests of the structure that will be discussed in Chapter 9. There was a small increase of the equivalent viscous damping in the diaphragm during Phase 10 with the installation of finish materials. Most of this increase was most likely a result of the gypsum wallboard that was installed on the first story ceiling (bottom of floor diaphragm).

7.3 Diaphragm Flexibility

Wood diaphragms can be classified as flexible or rigid depending on the ratio of the global diaphragm deflection at the center of the diaphragm, $\Delta_{global}$, to the shearwall deflection, $\Delta_{shearwall}$. The classification of diaphragms is important because the lateral forces in a structure are distributed differently for the two classifications of
diaphragms. For a flexible diaphragm, the diaphragm is modeled as a simple supported beam with the lateral forces in the walls based on tributary area of the diaphragm. However, for a rigid diaphragm, a torsional moment is developed when the centroid of the applied lateral forces does not coincide with the center of rigidity of the resisting shearwalls. The 1997 Edition of the Uniform Building Code (UBC Section 1630.6) states that “diaphragms shall be considered flexible for the purposes of distribution of story shear and torsional moment when the maximum lateral deformation of the diaphragm is more than two times the average story drift of the associated story.” Using this method of classification, the floor diaphragm was classified as either flexible or rigid using the global diaphragm deflection and the average shearwall deflection for each of the quasi-static tests. Table 7.2 shows the diaphragm to shearwall deflection ratio and the flexibility classifications.

### Table 7.2: Diaphragm Flexibility Classification

<table>
<thead>
<tr>
<th>Test Phase</th>
<th>Maximum Diaphragm Deflection, $\Delta_{\text{global}}$ (in)</th>
<th>Maximum Shearwall Deflection, $\Delta_{\text{shearwall}}$ (in)</th>
<th>Diaphragm to Shearwall Deflection Ratio, $\Delta_{\text{global}}/\Delta_{\text{shearwall}}$</th>
<th>Diaphragm Classification According to UBC 1630.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.167</td>
<td>0.064</td>
<td>2.61</td>
<td>Flexible</td>
</tr>
<tr>
<td>2</td>
<td>0.133</td>
<td>0.064</td>
<td>2.08</td>
<td>Flexible</td>
</tr>
<tr>
<td>3A</td>
<td>0.093</td>
<td>0.059</td>
<td>1.58</td>
<td>Rigid</td>
</tr>
<tr>
<td>3B</td>
<td>0.083</td>
<td>0.054</td>
<td>1.54</td>
<td>Rigid</td>
</tr>
<tr>
<td>4A</td>
<td>0.076</td>
<td>0.046</td>
<td>1.63</td>
<td>Rigid</td>
</tr>
<tr>
<td>4B</td>
<td>0.067</td>
<td>0.034</td>
<td>1.99</td>
<td>Rigid</td>
</tr>
<tr>
<td>4C</td>
<td>0.047</td>
<td>0.035</td>
<td>1.33</td>
<td>Rigid</td>
</tr>
<tr>
<td>4D</td>
<td>0.070</td>
<td>0.037</td>
<td>1.89</td>
<td>Rigid</td>
</tr>
<tr>
<td>5</td>
<td>0.029</td>
<td>0.023</td>
<td>1.26</td>
<td>Rigid</td>
</tr>
<tr>
<td>6</td>
<td>0.045</td>
<td>0.069</td>
<td>0.65</td>
<td>Rigid</td>
</tr>
<tr>
<td>7</td>
<td>0.029</td>
<td>0.068</td>
<td>0.43</td>
<td>Rigid</td>
</tr>
<tr>
<td>8</td>
<td>0.050</td>
<td>0.068</td>
<td>0.73</td>
<td>Rigid</td>
</tr>
<tr>
<td>9</td>
<td>0.049</td>
<td>0.073</td>
<td>0.68</td>
<td>Rigid</td>
</tr>
<tr>
<td>10</td>
<td>0.019</td>
<td>0.025</td>
<td>0.78</td>
<td>Rigid</td>
</tr>
</tbody>
</table>
Only the first two phases of testing with no blocking and no subfloor adhesive with 50% nailing and 100% nailing were classified as flexible diaphragms. Once the second story of the structure was constructed, the diaphragm to shearwall deflection ratio decreased further signifying an increase in rigidity of the diaphragm.

### 7.4 Diaphragm Deflection Using Code Equations

The 1997 Edition of the Uniform Build Code (ICBO, 1997) requires the deflection of a horizontal diaphragm to be calculated to check for excess deflections and for use in classification of diaphragm flexibility. The UBC separates the deflection of a horizontal diaphragm into four components: shear deformation, flexural deformation, nail slip, and chord splice slip. Note that the nail slip component of deflection is separate from the shear and flexural deformation according to the UBC method for computing deflections. In reality, the experimental shear and flexural deformations include the nail slip and cannot be separated. The UBC method assumes elastic beam theory for computing the shear and flexural deformations of a diaphragm.

For a diaphragm with a concentrated load applied at its center, the shear deformation is:

\[
\Delta_s = \frac{F \cdot L}{4G A_s}
\]  

(7.19)

where \( F \) is the applied concentrated load, \( L \) is the diaphragm length, \( G \) is the shear modulus of the plywood subfloor, and \( A_s \) is the area of shear of the plywood subfloor.

The flexural deformation of the diaphragm is:

\[
\Delta_f = \frac{F L^3}{48EI}
\]  

(7.20)
where $E$ is the elastic modulus of the diaphragm chord members and $I$ is the moment of inertia of the chord members computed from:

$$I = 2A_{chord} \left( \frac{d}{2} \right)^2$$  \hspace{1cm} (7.21)

where $A_{chord}$ is the area of the chord member (usually assumed to be the double top plates) and $d$ is the depth of the diaphragm. The nail slip deflection according to the UBC can be expressed as:

$$\Delta_n = 0.188Le_n$$  \hspace{1cm} (7.22)

where the variable $e_n$ represents the slip of the plywood sheathing with respect to the framing. UBC Table 23-2-K provides values for $e_n$ based on load per nail fastener. In addition, the nail deformation $e_n$ must be increased by 20% if Structural 1 sheathing is not used.

Using the maximum applied loads from the experimental data, a plywood shear modulus, $G$, of 90,000 psi, and a modulus of elasticity, $E$, for the chord members of 1,700,000 psi, the shear, flexural, and nail slip deflections were computed for the diaphragm. Note that chord splice slip deflection was not included since the chord members were continuous and not spliced in the test structure. Table 7.3 compares the computed deflections according to the UBC method with the experimental deflections.

Figure 7.7 shows plots of the experimental diaphragm deflection versus the deflection computed using the UBC method for shear deflection, flexural deflection, and total deflection. In each of these graphs, a 45-degree line is shown representing perfect correlation between the experimental results and the UBC predictions. Note that the experimental shear and flexural deflections include nail slip deflection, while
the shear and flexural deflections computed using the UBC method do not. Since the majority of the shear deflection data points are to the right of the 45-degree line, the UBC method over predicts the shear deflection. If the nail-slip deflection were to be included in the UBC shear deflection equation, the shear deflection according to UBC method would be even farther from the experimental shear deflection. As shown in the flexural deflection comparison graph of Figure 7.7, the UBC method underestimates the flexural deflection. However, if the nail-slip deflection were to be included in the UBC flexural deflection equation, the flexural deflection predicted by the UBC method would be much closer to the experimental deflection. A comparison of the total experimental deflection and the total deflection according to the UBC method in Figure 7.7 shows that the UBC method predicts the total deflection reasonably well for some configurations, but poorly for others. This was probably the result of the UBC method using the number of nails as the only variable in the nail slip component of

Table 7.3: Comparison of Experimental and UBC Method Diaphragm Deflections

<table>
<thead>
<tr>
<th>Test Phase</th>
<th>UBC Method</th>
<th>Experimental</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear Deflection, $\Delta_s$ (in)</td>
<td>Flexural Deflection, $\Delta_f$ (in)</td>
</tr>
<tr>
<td>1</td>
<td>0.042</td>
<td>0.008</td>
</tr>
<tr>
<td>2</td>
<td>0.045</td>
<td>0.008</td>
</tr>
<tr>
<td>3A</td>
<td>0.042</td>
<td>0.008</td>
</tr>
<tr>
<td>3B</td>
<td>0.041</td>
<td>0.008</td>
</tr>
<tr>
<td>4A</td>
<td>0.032</td>
<td>0.006</td>
</tr>
<tr>
<td>4B</td>
<td>0.037</td>
<td>0.007</td>
</tr>
<tr>
<td>4C</td>
<td>0.041</td>
<td>0.008</td>
</tr>
<tr>
<td>4D</td>
<td>0.034</td>
<td>0.006</td>
</tr>
<tr>
<td>5</td>
<td>0.036</td>
<td>0.007</td>
</tr>
<tr>
<td>6</td>
<td>0.041</td>
<td>0.008</td>
</tr>
<tr>
<td>7</td>
<td>0.031</td>
<td>0.006</td>
</tr>
<tr>
<td>8</td>
<td>0.041</td>
<td>0.008</td>
</tr>
<tr>
<td>9</td>
<td>0.037</td>
<td>0.007</td>
</tr>
<tr>
<td>10</td>
<td>0.051</td>
<td>0.010</td>
</tr>
</tbody>
</table>
deflection. It does not consider diaphragm blocking, gluing, construction of boundary elements, and finish materials.

7.5 Phase Lag Between Shear and Global Diaphragm Deformation

A lag in the shear deformation behind the global diaphragm deformation was observed at low levels of diaphragm deformation. One would expect to find a plot of diaphragm shear deformation versus global diaphragm deformation to be linear, but it was found a hysteretic behavior was exhibited as shown in Figure 7.8 for Phase 5
testing. Note that the trajectory of these hysteresis loops is counterclockwise. Similar plots for each phase of quasi-static testing can be found in Appendix E.

A simple mechanical model of the diaphragm can explain this lag in shear deformation with a flexural spring in series with parallel shear slider and shear springs as shown in Figure 7.9. The flexural spring of this model represents the flexural stiffness of the diaphragm. The shear spring of the model represents the shear stiffness of the diaphragm. The shear slider represents shear friction that must be overcome prior to activating the shear spring.

![Figure 7.8: Hysteretic Behavior Showing Shear Phase Lag for Phase 5 Testing](image)
Each point of the shear versus global deformation hysteresis can be predicted using this mechanical model as shown in Figure 7.10. The shear slider is locked between Point 0 and Point 1 preventing shear deformation. As a result, the global deformation is equal to the flexural deformation for this stage. The shear slider unlocks (shear friction is overcome) allowing the shear spring to deform between Point 1 and Point 2. The global deformation of this stage is equal to the sum of the flexural deformation and shear deformation. At Point 2, the diaphragm loading direction is changed. The shear slider locks again preventing the shear spring from deforming while the flexural spring begins to release tension between Point 2 and Point 3. At Point 3, the flexural spring is relaxed. Between Point 3 and Point 4, the flexural spring begins to compress while the shear slider is still locked. Once the shear slider reaches the friction load at Point 4, the shear spring becomes active in compression with the flexural spring. At Point 5, the shear spring has returned to its relaxed position while the flexural spring is compressed due to this phase lag.
At Point 6, the diaphragm is loaded in the opposite direction causing the shear slider to lock once again preventing the shear spring from deforming. The flexural spring decompresses between Point 6 and Point 7 returning to its relaxed position. The flexural spring continues to elongate to Point 8 when the shear slider unlocks as the shear friction load has been reached. Both the shear and flexural springs are active returning to Point 1 as the cycle is continued again. Comparing the experimental hysteresis from Phase 5 in Figure 7.8 with the theoretical hysteresis using the mechanical model in Figure 7.10, the experimental hysteresis exhibits a more rounded shape. This discrepancy suggests that the diaphragm does not behave exactly as the
mechanical model. The shear friction in the diaphragm is gradually overcome instead of being overcome instantaneously as assumed in the mechanical model.

7.6 Conclusions and Recommendations
Several important conclusions and recommendations can be made regarding the results of the quasi-static tests that were performed. The nailed diaphragm exhibited a hysteretic response even at low levels of deformation. The first quasi-static tests showed that the nailing schedule (50% or 100%) had little influence on the stiffness of the diaphragm. However, the installation of diaphragm blocking at unsupported plywood panel edges and subfloor adhesive caused a significant increase in the diaphragm shear stiffness. The influence of blocking was more profound on the diaphragm shear stiffness for the diaphragms without subfloor adhesive compared to those with subfloor adhesive. The diaphragm nailing, blocking, and gluing had little influence on the diaphragm flexural stiffness since the flexural stiffness is commonly thought to be a function of the chord members. However, there was a significant increase in the flexural stiffness of the diaphragm once the second story was constructed. The second story walls may have acted as additional chord members for the diaphragm causing the increase in flexural stiffness.

The UBC method for computing diaphragm deflection predicted the diaphragm deflection well for some configurations and poorly for other configurations. The poor predictions are a result of the equations not considering diaphragm blocking, gluing, boundary elements, and finish materials. According to the UBC flexibility classifications for diaphragms, most of the diaphragm configurations tested behaved
rigidly. Note that current design practice often assumes a flexible diaphragm for the lateral load distribution in a woodframe structure.
Chapter 8  Results of Frequency Evaluation Tests

The purpose of the frequency evaluation tests was to identify the fundamental natural frequency and mode shape of the test structure before and after each seismic test. The procedure used to develop the frequency test protocol and the data analysis procedure is discussed. The fundamental natural frequency results for each of the frequency evaluation tests are presented in terms of the variation of fundamental frequency and variation of normalized lateral stiffness. Lastly, the structural mode shapes before and after a particular phase of seismic tests are compared.

8.1  Testing Procedure

For each of the frequency evaluation tests, the test structure was excited using
a low-amplitude 1 – 20 Hz, clipped-band, flat white noise. The Root Mean Square (RMS) amplitude of the input motion was between 0.025 g and 0.040 g. A power spectral density plot of the input acceleration for one of the shake table tests is shown below in Figure 8.1. Note that the input energy is relatively constant between frequencies of 1 and 20 Hz. A dedicated ambient vibration analysis software package (Experimental Dynamics Investigations, 1993) was used to determine the fundamental natural frequency of the test structure from power spectral density plots of absolute acceleration records at various locations throughout the test structure.

Prior to conducting the ambient vibrations tests, several parameters that describe the test protocol were developed. The Nyquist frequency, $f_{Nyquist}$, is the maximum frequency that can be extracted from a data signal during an ambient vibration test. The Nyquist frequency is one-half of the data sampling

![Figure 8.1: Power Spectral Density Plot of Input Shake Table Acceleration for Frequency Test 5.F.0](image)
frequency, or in terms of the data sampling rate, the Nyquist frequency is given by:

\[ f_{\text{Nyquist}} = \frac{1}{2\Delta t} \]  

(8.1)

where \( \Delta t \) is the data sampling rate in seconds during the test. As a result, the sampling rate must be selected such that the highest expected frequency of the data signal is less than the Nyquist frequency. Aliasing problems could occur if the data signal contains frequencies exceeding the Nyquist frequency.

The acceleration data signal in time domain is divided into \( N_w \) windows with each window having a time duration of \( s_o \) as shown in Figure 8.2. The total duration of \( N_w \) windows of duration \( s_o \) is given by:

\[ S_{\text{total}} = N_w \cdot s_o \]  

(8.2)
The number of data points per window, $N$, is given by:

$$N = \frac{s_o}{\Delta t}$$  \hspace{1cm} (8.3)

where $N$ must be an integer of magnitude $2^x$ where $x$ is any positive integer. The frequency resolution, $\Delta f$ is dependent on the time duration of each window:

$$\Delta f = \frac{1}{s_o}$$  \hspace{1cm} (8.4)

These relationships were used to determine the appropriate test protocol parameters for the ambient vibration testing of the woodframe test structure. Table 8.1 shows the parameters for the ambient vibration tests conducted on the test structure.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nyquist Frequency, $f_{\text{Nyquist}}$</td>
<td>100 Hz</td>
</tr>
<tr>
<td>Data Sampling Rate, $\Delta t$</td>
<td>0.005 sec</td>
</tr>
<tr>
<td>Number of Points Per Sampling Window, $N$</td>
<td>4096</td>
</tr>
<tr>
<td>Duration of Each Sampling Window, $s_o$</td>
<td>20.48</td>
</tr>
<tr>
<td>Frequency Resolution, $\Delta f$</td>
<td>0.0488 Hz</td>
</tr>
<tr>
<td>Number of Sampling Windows, $N_w$</td>
<td>8</td>
</tr>
<tr>
<td>Total Duration, $S_{\text{total}}$</td>
<td>163.84 sec</td>
</tr>
</tbody>
</table>

### 8.2 Data Analysis

The ambient vibration analysis software package U2 (Experimental Dynamics Investigations, 1993) was used to identify the fundamental natural frequency, the structural mode shape, and the phase angle between channels for each of the frequency tests. Six acceleration time-histories throughout the structure were used to identify the dynamic characteristics of the test structure. The fundamental natural frequency of the structure was determined from the power spectral density of each of the six
acceleration records. In order to determine the power spectral density of each acceleration record, each window of duration $s_o$ is transformed from time domain into frequency domain using a Fast Fourier Transform (FFT). The resulting Fourier transform, $F(\omega)$, is composed of a real part, $a(\omega)$, and an imaginary part $b(\omega)$:

$$F(\omega) = a(\omega) + i b(\omega)$$

(8.5)

The magnitude of the Fourier transform or Fourier spectrum is computed for each of the $N_w$ windows using:

$$|F(\omega)| = |a(\omega)|^2 + |b(\omega)|^2$$

(8.6)

The power spectral density of the acceleration record is estimated as the mean of the Fourier spectra, $S(\omega)$, taken across for each of the $N_w$ windows. The fundamental natural frequency of the test structure is then estimated as the frequency corresponding to the peak spectral density. Figure 8.3 outlines the process used in developing the

![Figure 8.3: Development of Power Spectral Density](image-url)
power spectral density and fundamental natural frequency for each acceleration record. Only the fundamental natural frequency was determined for the test structure, but higher mode frequencies were also observed using this procedure.

The mode shape for the test structure can be determined using the peak values of the spectral density functions at the fundamental natural frequency. One channel of acceleration is selected as the reference channel and is assigned a modal value of 1.0. The modal values for each of the other acceleration records are then computed relative to the reference channel. If $\alpha$ is the peak spectral density amplitude at the natural frequency for the reference channel, and $\beta$ is the peak spectral density amplitude at the same natural frequency for another channel $i$, the modal value corresponding to this other channel, $A_i$, is given by:

$$A_i = \frac{\beta}{\sqrt{\alpha}}$$  \hspace{1cm} (8.7)

The same procedure can be used to compute the modal values for each of the other desired acceleration channels.

The phase angle for a particular acceleration record is computed using the real part and imaginary part of the frequency response function developed for each of the $N_w$ windows. Similar to the power spectral density computation, the real part and the imaginary part used in computing the phase angle is the mean of the frequency response values of each of the $N_w$ number of windows. The phase angle for a particular acceleration record, $\phi(\omega)$, is given by:

$$\phi(\omega) = \tan^{-1}\left(\frac{-b(\omega)}{a(\omega)}\right)$$  \hspace{1cm} (8.8)
where $a(\omega)$ is the mean of the real part of the frequency response functions and $b(\omega)$ is the mean of the imaginary part of the frequency response functions. However, of primary importance is the phase difference between two channels of acceleration. If two channels of acceleration have a phase angle difference of 0 degree at the natural frequency, their peak modal values are in phase. If two channels of acceleration have a phase angle difference of 180 degrees at the natural frequency, their peak modal values are out of phase.

8.3 Variation of Natural Frequency

Six accelerometers at the second floor level, roof eaves, and roof ridge on both the east and west wall elevations were used to identify the North-South fundamental natural frequency of the test structure. Appendix F shows the power spectral density functions for each of the six accelerometers and for each of the frequency evaluation tests that were conducted. Table 8.2 shows a summary of the North-South fundamental frequencies of the test structure. From this data, it can be seen that the natural frequency ranges from 2.93 Hz following the Level 5 seismic test of Phase 9 to 6.49 Hz prior to the Level 1 seismic test of Phase 10. Figure 8.4 shows the variations of the North-South fundamental frequencies with respect to the peak ground acceleration of the input motion. The North-South fundamental frequencies of the fully-sheathed Phase 5 test structure are considerably higher than the fundamental frequencies of Phases 6, 7, and 8 with small openings. The small variations in natural frequencies for Phases 6, 7, and 8 suggests that perforated shearwall design and “conventional” construction do not have a significant influence on the fundamental frequency of the test structure when compared to the engineered test
Table 8.2: Summary of North-South Fundamental Frequencies

<table>
<thead>
<tr>
<th>Test Phase</th>
<th>Structure Configuration</th>
<th>Test Level</th>
<th>Test Designation</th>
<th>Fundamental Frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Two stories, fully sheathed, 100% nailing, no blocking, PL400 adhesive</td>
<td>0</td>
<td>5.F.0</td>
<td>5.62</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>5.F.1</td>
<td>5.42</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>5.F.2</td>
<td>5.37</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>5.F.3</td>
<td>5.37</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>5.F.4</td>
<td>5.13</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4R</td>
<td>5.F.4R</td>
<td>4.98</td>
</tr>
<tr>
<td>6</td>
<td>Two stories, small openings, 100% nailing, no blocking, PL400 adhesive</td>
<td>0</td>
<td>6.F.0</td>
<td>4.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>6.F.1</td>
<td>4.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>6.F.2</td>
<td>4.05</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>6.F.3</td>
<td>4.05</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>6.F.4</td>
<td>3.66</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4R</td>
<td>6.F.4R</td>
<td>3.71</td>
</tr>
<tr>
<td>6A</td>
<td>Two stories, small openings with wastewall sheathing removed, 100% nailing, no blocking, PL400 adhesive</td>
<td>2</td>
<td>6A.F.2</td>
<td>3.27</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>6A.F.3</td>
<td>3.22</td>
</tr>
<tr>
<td>7</td>
<td>Two stories, perforated shearwall design, 100% nailing, no blocking, PL400 adhesive</td>
<td>0</td>
<td>7.F.0</td>
<td>3.91</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>7.F.1</td>
<td>3.91</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>7.F.2</td>
<td>3.91</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>7.F.3</td>
<td>3.66</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3R</td>
<td>7.F.3R</td>
<td>3.56</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>7.F.4</td>
<td>3.17</td>
</tr>
<tr>
<td>7A</td>
<td>Two stories, perforated shearwall design with anchor bolts installed adjacent to door openings, 100% nailing, no blocking, PL400 adhesive</td>
<td>2</td>
<td>7A.F.2</td>
<td>3.37</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>7A.F.3</td>
<td>3.37</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>7A.F.4</td>
<td>3.37</td>
</tr>
<tr>
<td>8</td>
<td>Two stories, conventional construction, 100% nailing, no blocking, PL400 adhesive</td>
<td>0</td>
<td>8.F.0</td>
<td>4.15</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>8.F.1</td>
<td>4.05</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>8.F.2</td>
<td>4.05</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>8.F.3</td>
<td>3.91</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3R</td>
<td>8.F.3R</td>
<td>3.81</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>8.F.4</td>
<td>3.47</td>
</tr>
<tr>
<td>9</td>
<td>Two stories, large opening, 100% nailing, no blocking, PL400 adhesive</td>
<td>0</td>
<td>9.F.0</td>
<td>3.96</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>9.F.1</td>
<td>3.91</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>9.F.2</td>
<td>3.71</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>9.F.3</td>
<td>3.66</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3R</td>
<td>9.F.3R</td>
<td>3.42</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>9.F.4</td>
<td>2.93</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>9.F.5</td>
<td>2.93</td>
</tr>
<tr>
<td>10</td>
<td>Two stories, finish materials, 100% nailing, no blocking, PL400 adhesive</td>
<td>0</td>
<td>10.F.0</td>
<td>6.49</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>10.F.1</td>
<td>6.35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>10.F.2</td>
<td>6.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>10.F.3</td>
<td>5.76</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>10.F.4</td>
<td>5.71</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4R</td>
<td>10.F.4R</td>
<td>5.47</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>10.F.5</td>
<td>5.37</td>
</tr>
</tbody>
</table>
structure of Phase 6. This can be expected since the frequency evaluation tests were performed at low amplitude. The large opening on the east wall of the Phase 9 test structure slightly reduced the fundamental frequencies from that of Phases 6, 7, and 8. The fundamental frequencies for Phase 10 are significantly higher than the fully-sheathed Phase 5 test structure as a result of the finish materials. This suggests that finish materials should be included when computing the fundamental frequency of a woodframe structure. Over the sequence of testing for each phase, there was a distinct
decrease in fundamental frequencies resulting from damage to the test structure. It can also be seen that the largest reduction in the fundamental frequencies occurs following the Level 4 seismic test. However, there was little reduction in the fundamental frequencies between Level 4 and Level 5.

8.4 Variation of Normalized Lateral Stiffness

Using a single-degree-of-freedom assumption, the variation of normalized lateral stiffness can be obtained for each phase of testing. The fundamental frequency of the test structure at Level \( n \) of testing is approximated by:

\[
\omega_n = \sqrt{\frac{k_n}{m}}
\]  

(8.9)

where \( k_n \) is the lateral stiffness of the SDOF system following seismic test Level \( n \) and \( m \) is the mass of the SDOF system. Squaring both sides of Equation 8.10 gives:

\[
\omega_n^2 = \frac{k_n}{m}
\]  

(8.10)

Similarly, the square of the initial natural frequency prior to seismic testing, \( \omega_o \), is given by:

\[
\omega_o^2 = \frac{k_o}{m}
\]  

(8.11)

where \( k_o \) is the initial lateral stiffness. Dividing Equation 8.12 by Equation 8.11 gives:

\[
\frac{k_n}{k_o} = \frac{\omega_n^2}{\omega_o^2}
\]  

(8.12)

Equation 8.12 can be used to compute the normalized lateral stiffness of the test structure for a particular test Level \( n \) relative to the initial lateral stiffness of the structure for that phase of testing. Table 8.3 shows a summary of the normalized lateral stiffness for the various phases of testing. Note that the normalized lateral
stiffness for Level 0 of each phase has been set at 100%. A plot showing the variation of normalized lateral stiffness with peak ground acceleration of the input motion is presented in Figure 8.5.
Table 8.3: Summary of Normalized Lateral Stiffness of Test Structure

<table>
<thead>
<tr>
<th>Test Phase</th>
<th>Structure Configuration</th>
<th>Test Level</th>
<th>Test Designation</th>
<th>Normalized Lateral Stiffness (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Two stories, fully sheathed, 100% nailing, no blocking, PL400 adhesive</td>
<td>0</td>
<td>5.F.0</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>5.F.1</td>
<td>93</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>5.F.2</td>
<td>91</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>5.F.3</td>
<td>91</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>5.F.4</td>
<td>83</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4R</td>
<td>5.F.4R</td>
<td>79</td>
</tr>
<tr>
<td>6</td>
<td>Two stories, small openings, 100% nailing, no blocking, PL400 adhesive</td>
<td>0</td>
<td>6.F.0</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>6.F.1</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>6.F.2</td>
<td>91</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>6.F.3</td>
<td>91</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>6.F.4</td>
<td>74</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4R</td>
<td>6.F.4R</td>
<td>76</td>
</tr>
<tr>
<td>7</td>
<td>Two stories, perforated shearwall design, 100% nailing, no blocking, PL400 adhesive</td>
<td>0</td>
<td>7.F.0</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>7.F.1</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>7.F.2</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>7.F.3</td>
<td>88</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3R</td>
<td>7.F.3R</td>
<td>83</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>7.F.4</td>
<td>66</td>
</tr>
<tr>
<td>8</td>
<td>Two stories, conventional construction, 100% nailing, no blocking, PL400 adhesive</td>
<td>0</td>
<td>8.F.0</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>8.F.1</td>
<td>95</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>8.F.2</td>
<td>95</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>8.F.3</td>
<td>89</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3R</td>
<td>8.F.3R</td>
<td>84</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>8.F.4</td>
<td>70</td>
</tr>
<tr>
<td>9</td>
<td>Two stories, large opening, 100% nailing, no blocking, PL400 adhesive</td>
<td>0</td>
<td>9.F.0</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>9.F.1</td>
<td>97</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>9.F.2</td>
<td>88</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>9.F.3</td>
<td>85</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3R</td>
<td>9.F.3R</td>
<td>75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>9.F.4</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>9.F.5</td>
<td>55</td>
</tr>
<tr>
<td>10</td>
<td>Two stories, finish materials, 100% nailing, no blocking, PL400 adhesive</td>
<td>0</td>
<td>10.F.0</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>10.F.1</td>
<td>96</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>10.F.2</td>
<td>88</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>10.F.3</td>
<td>79</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>10.F.4</td>
<td>77</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4R</td>
<td>10.F.4R</td>
<td>71</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>10.F.5</td>
<td>68</td>
</tr>
</tbody>
</table>

The majority of the stiffness degradation takes place with the Level 4 seismic test at a peak ground acceleration of 0.50 g and corresponding to a hazard level of
10% probability of exceedance in 50 years. The lateral stiffness degraded to 79% and 76% of the initial lateral stiffness for Phase 5 and Phase 6, respectively, suggesting the presence of small symmetric openings did not influence the stiffness degradation. However, the lateral stiffness of the test structure degraded to 66% of the initial lateral stiffness using perforated shearwall design of Phase 7. However, this increased stiffness degradation may have been the result of not replacing the east and west wall sheathing between Phase 6 and Phase 7. The test structure using “conventional” construction (Phase 8) had final lateral stiffness of 70% of the initial lateral stiffness, which is slightly lower than the engineered structure of Phase 6. Once the large opening was introduced to the first story east wall during Phase 9, the stiffness degradation worsened with a final lateral stiffness of only 55% of the initial lateral stiffness. This suggests that woodframe structure with significant eccentricity may require significant repairs to the lateral force resisting system following a large earthquake. The exterior stucco and interior gypsum wallboard installed on the test structure slowed the degradation of the lateral force resisting system when compared to the stiffness degradation of Phase 9. Similar to the fundamental frequencies, there was little reduction the normalized lateral stiffness between Level 4 and Level 5.

8.5 Variation of Mode Shapes

The variation of mode shapes in the structure between levels of testing and changes in structural configuration can be used to identify relative shearwall stiffness, shearwall stiffness degradation, and torsion in the structure. The structural mode shapes for each of the frequency evaluation tests conducted are shown in Appendix F.
The mode shape of the structure at the start of Phase 5 is nearly linear between the base and roof eaves suggesting that the initial stiffness of the first and second story fully sheathed shearwalls is approximately the same. As shown by an increase in the modal value at the floor level at the end of Phase 5, the first story shearwall stiffness was degraded more than the second story stiffness. With the small door and window openings of Phase 6, the initial mode shape was nearly linear suggesting the initial stiffness of the first and second story shearwalls were relatively equal. There was slightly more degradation of the first story shearwalls than the second story shearwalls over the duration of Phase 6 testing as shown by an increase in the modal value at the floor level. The first story shearwalls of Phase 7 with the perforated shearwall design had increased initial modal value at the floor level compared to that of Phase 5 and Phase 6, but this may have been a result of not replacing the east and west wall sheathing between Phase 6 and Phase 7. Again, there was slightly more degradation of the first story shearwalls than the second story shearwalls during Phase 7. An initial linear mode shape for Phase 8 with conventional construction suggests an approximate equal initial stiffness of the first and second story shearwalls. Following these tests, there was an approximately equal degradation in stiffness of the first and second story shearwalls. Note that for each of these tests discussed thus far, the torsional response shown by the mode shapes was negligible.

Torsion became very evident in the structure as shown by the mode shapes for Phase 9 and Phase 10, with the addition of the large opening in the first story east shearwall. Figure 8.6 shows a floor level modal value of 0.57 at the east shearwall compared to only 0.38 at the west shearwall. Note that the modal values at the roof
ridge for both the east and west wall elevations remained nearly the same suggesting the torsion is not transferred up through the structure.

**Before Level 1**

**After Level 5**

Figure 8.6: Variation of Structural Mode Shape for Phase 9
After the structure underwent the six seismic tests of Phase 9, the structural mode shape changed considerably. The modal value at the floor level of the east wall only increased slightly from 0.57 to 0.62 suggesting the degradation in the east shearwall stiffness was small. However, the modal value at the floor level of the west wall increased considerably from 0.38 to 0.53 suggesting a significant stiffness degradation of the west shearwall. Since the west shearwall was initially much stiffer than the east shearwall, it attracted more seismic load and therefore was degraded more than the east shearwall.

The test structure exhibited even more torsional response during Phase 10 than Phase 9 as evidenced by the modal values at the floor level of the east shearwall as shown in Figure 8.7. Again, the modal value at the roof ridge is nearly identical for the east and west wall elevations.

Unlike the significant degradation of the west shearwall during Phase 9 testing, both the east and west first story shearwalls showed little stiffness degradation during Phase 10. The modal values at the floor level for both the east and west shearwalls are only slightly higher than the initial modal values. This suggests that the structure behaved nearly elastically with the addition of wall finish materials.
Before Level 1

East Elevation

D18 0.98

D8 0.75

D23 1.00

North

After Level 5

East Elevation

D18 0.99

D8 0.76

D23 1.00

North

D14 0.80

D4 0.45

D19 0.99

North

Figure 8.7: Variation of Structural Mode Shape for Phase 10
Chapter 9  Results of Damping Evaluation Tests

The main objective of the damping evaluation tests was to quantify the first modal equivalent critical viscous damping in the test structure before and after each seismic test. The testing procedure is outlined for the damping evaluation tests. In addition, the data analysis procedure used to determine the damping ratios from the experimental data is described. Results of the damping evaluation tests are presented for each of the damping evaluation tests.

9.1  Testing Procedure

For each of the damping evaluation tests, the test structure was excited in resonance by the shake table with a sinusoidal acceleration at the fundamental

154
frequency of the test structure found from the frequency evaluation tests. The peak acceleration at the roof level varied between damping evaluation tests, but was typically around 0.05 g. Once resonance occurred in the test structure, the structure was excited for an additional 30 seconds and then the table was suddenly stopped with a stop ramp rate of 1,000,000% which corresponds to the table stopping in 1/10,000 of a second. The acceleration data for the damping evaluation tests was filtered at a frequency 10% greater than the fundamental frequency of the test structure for a particular test to eliminate higher mode effects.

9.2 Data Analysis

The logarithmic decrement procedure was used to determine the first modal viscous damping ratio for each of the damping evaluation tests. The logarithmic decrement procedure is derived from the displacement of a single-degree-of-freedom system undergoing viscously damped free vibration, $u(t)$ (Filiatrault, 1998):

$$u(t) = R \cdot e^{-\xi \omega_d t} \sin(\omega_d t + \phi)$$

where $\xi$ is the viscous damping ratio, $\omega_n$ is the undamped natural frequency of the system, $\omega_d$ is the damped natural frequency of the system, and $R$ and $\phi$ are constants representing the initial conditions. The velocity can be determined from the displacement equation by differentiating once:

$$\dot{u}(t) = R \cdot e^{-\xi \omega_d t} \left( \omega_d \cos(\omega_d t + \phi) - \xi \omega_n \sin(\omega_d t + \phi) \right)$$

The acceleration can be determined from the velocity equation by differentiating again:

$$\ddot{u}(t) = R \cdot e^{-\xi \omega_d t} \left( \xi^2 \omega_n^2 \sin(\omega_d t + \phi) - 2\xi \omega_n \omega_d \cos(\omega_d t + \phi) - \omega_d^2 \sin(\omega_d t + \phi) \right)$$
For the logarithmic decrement procedure, the ratio of successive cycles of acceleration is given by:

$$\frac{\ddot{u}_i}{\ddot{u}_{i+1}} = \frac{\ddot{u}(t)}{\ddot{u}(t + T_d)}$$ (9.4)

where $T_d$ is the damped natural period of the system given by:

$$T_d = \frac{2\pi}{\omega_d}$$ (9.5)

Substituting Equation 9.3 into Equation 9.4 and using Equation 9.5 for the damped natural period gives:

$$\frac{\ddot{u}_i}{\ddot{u}_{i+1}} = \frac{R \cdot e^{-\xi \omega_d t}}{R \cdot e^{-\xi \omega_d \left(t + \frac{2\pi}{\omega_d}\right)}}$$ (9.6)

Simplifying Equation 9.6 gives:

$$\frac{\ddot{u}_i}{\ddot{u}_{i+1}} = e^{\frac{2\pi \xi \omega_d}{\omega_d}}$$ (9.7)

The damped natural frequency can be expressed as:

$$\omega_d = \omega_n \sqrt{1 - \xi^2}$$ (9.8)

Substituting Equation 9.8 into Equation 9.7 gives:

$$\frac{\ddot{u}_i}{\ddot{u}_{i+1}} = e^{\frac{2\pi \xi}{\sqrt{1 - \xi^2}}}$$ (9.9)

Equation 9.9 represents the ratio of successive peaks of acceleration of either positive acceleration peaks or negative acceleration peaks. However, both positive and negative acceleration peaks can be taken into account using the differential acceleration as shown in Figure 9.1.
The ratio of successive peak-to-peak acceleration is given by:

$$\frac{\Delta u_i}{\Delta u_{i+1}} = e^{\frac{2\pi \xi}{\sqrt{1-\xi^2}}} \quad (9.10)$$

By taking the natural logarithm of this ratio of Equation 9.10, the logarithmic decrement, $\delta$, is given by:

$$\delta = \ln\left(\frac{\Delta u_i}{\Delta u_{i+1}}\right) = \frac{2\pi \xi}{\sqrt{1-\xi^2}} \quad (9.11)$$

Since most structures have low damping, it can be assumed that:

$$\sqrt{1-\xi^2} \approx 1 \quad (9.12)$$

Therefore, the logarithmic decrement simplifies to:

$$\delta \equiv 2\pi \xi \quad (9.13)$$

The viscous damping can then be computed using the peak-to-peak acceleration of successive cycles:

$$\zeta = \frac{\ln\left(\frac{\Delta u_i}{\Delta u_{i+1}}\right)}{2\pi} \quad (9.14)$$
For the experimental damping tests performed, the viscous damping was computed using the recorded acceleration response at several locations in the structure. Since the test structure was excited at its fundamental natural frequency, it was necessary to exclude the cycles of forced vibration when computing the logarithmic decrement. A cycle was identified as the first cycle of free vibration once the peak-to-peak displacement for the first cycle, $\Delta u_j$, was less than 85% of the maximum peak-to-peak acceleration occurring during forced vibration, $\Delta \ddot{u}_{\text{forced}}$. Then, the logarithmic decrement and the viscous damping ratio were computed using the peak-to-peak accelerations of the first two cycles. Similarly, the logarithmic decrement and the viscous damping ratio were computed for successive cycles (cycles two and three, cycles three and four, etc.). However, if the amplitude of the peak-to-peak acceleration for cycle $i + 1$ was greater than the amplitude of the peak-to-peak acceleration for cycle $i$, the viscous damping was computed using cycle $i$ and cycle $i + 2$:

$$\zeta = \frac{\ln \left( \frac{\Delta \ddot{u}_i}{\Delta \ddot{u}_{i+2}} \right)}{2\pi \cdot 2}$$  \hspace{1cm} (9.15)

If the amplitude of the peak-to-peak acceleration of cycle $i + 1$ as well as cycle $i + 2$ was greater than the amplitude of the differential acceleration of cycle $i$, the logarithmic decrement procedure was stopped for that channel of acceleration. The average viscous damping for each channel of acceleration was computed by averaging the viscous damping ratios for all cycles.
9.3 Damping Test Results

The mean damping ratio for each of the damping evaluation tests was computed using 9 accelerometers oriented in the North-South direction in the test structure. The 9 accelerometers were located at the west wall, middle of the structure, and at the east wall at the floor level, roof eaves, and roof ridge. The acceleration time-histories for each of these accelerometers obtained during the damping tests are shown in Appendix G along with the corresponding percentage of critical damping for each channel. Note that the peak acceleration recorded at each of the nine accelerometers is relatively constant for each of the damping evaluation tests.

The mean value of damping for these nine accelerometers computed for each of the damping evaluation tests are summarized in Table 9.1. Figure 9.2 shows the

![Figure 9.2: Distribution of Percentage of Critical Damping](image-url)
Table 9.1: Summary of Mean Percentage of Critical Damping

<table>
<thead>
<tr>
<th>Test Phase</th>
<th>Structure Configuration</th>
<th>Test Level</th>
<th>Test Designation</th>
<th>Mean Percentage of Critical Damping</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Two stories, fully sheathed, 100% nailing, no blocking, PL400 adhesive</td>
<td>0</td>
<td>5.D.0</td>
<td>4.7%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>5.D.1</td>
<td>5.7%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>5.D.2</td>
<td>12.8%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>5.D.3</td>
<td>9.6%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>5.D.4</td>
<td>12.9%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4R</td>
<td>5.D.4R</td>
<td>11.4%</td>
</tr>
<tr>
<td>6</td>
<td>Two stories, small openings, 100% nailing, no blocking, PL400 adhesive</td>
<td>0</td>
<td>6.D.0</td>
<td>5.5%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>6.D.1</td>
<td>10.8%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>6.D.2</td>
<td>6.4%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>6.D.3</td>
<td>7.2%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>6.D.4</td>
<td>7.0%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4R</td>
<td>6.D.4R</td>
<td>10.2%</td>
</tr>
<tr>
<td>6A</td>
<td>Two stories, small openings with 'wastewall' sheathing removed, 100% nailing, no blocking, PL400 adhesive</td>
<td>2</td>
<td>6A.D.2</td>
<td>6.5%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>6A.D.3</td>
<td>7.4%</td>
</tr>
<tr>
<td>7</td>
<td>Two stories, perforated shearwall design, 100% nailing, no blocking, PL400 adhesive</td>
<td>0</td>
<td>7.D.0</td>
<td>8.6%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>7.D.1</td>
<td>9.2%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>7.D.2</td>
<td>10.0%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>7.D.3</td>
<td>7.1%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3R</td>
<td>7.D.3R</td>
<td>8.3%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>7.D.4</td>
<td>8.3%</td>
</tr>
<tr>
<td>7A</td>
<td>Two stories, perforated shearwall design with anchor bolts installed adjacent to door openings, 100% nailing, no blocking, PL400 adhesive</td>
<td>2</td>
<td>7A.D.2</td>
<td>7.7%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>7A.D.3</td>
<td>6.7%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>7A.D.4</td>
<td>7.3%</td>
</tr>
<tr>
<td>8</td>
<td>Two stories, conventional construction, 100% nailing, no blocking, PL400 adhesive</td>
<td>0</td>
<td>8.D.0</td>
<td>6.3%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>8.D.1</td>
<td>7.9%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>8.D.2</td>
<td>9.4%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>8.D.3</td>
<td>10.5%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3R</td>
<td>8.D.3R</td>
<td>8.3%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>8.D.4</td>
<td>7.1%</td>
</tr>
<tr>
<td>9</td>
<td>Two stories, large opening, 100% nailing, no blocking, PL400 adhesive</td>
<td>0</td>
<td>9.D.0</td>
<td>4.2%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>9.D.1</td>
<td>4.3%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>9.D.2</td>
<td>4.2%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>9.D.3</td>
<td>3.9%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3R</td>
<td>9.D.3R</td>
<td>6.2%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>9.D.4</td>
<td>7.2%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>9.D.5</td>
<td>8.7%</td>
</tr>
<tr>
<td>10</td>
<td>Two stories, finish materials, 100% nailing, no blocking, PL400 adhesive</td>
<td>0</td>
<td>10.D.0</td>
<td>3.3%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>10.D.1</td>
<td>6.1%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>10.D.2</td>
<td>11.5%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>10.D.3</td>
<td>8.1%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>10.D.4</td>
<td>7.4%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4R</td>
<td>10.D.4R</td>
<td>5.9%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>10.D.5</td>
<td>6.2%</td>
</tr>
</tbody>
</table>
percentage of critical damping for each of the damping evaluation tests plotted versus the input peak ground acceleration. Lines showing the mean as well as mean plus one standard deviation and mean minus one standard deviation of damping are shown on the graph. The distribution of the percentage of critical damping is scattered for each of the damping evaluation tests. However, the majority of damping values are close to the mean percentage of critical damping of 7.6%. Most of the damping values are bounded by the mean plus one standard deviation and the mean minus one standard deviation (5.3% to 10.0% of critical).
Chapter 10 Results of Seismic Tests

Seismic tests were performed on the test structure in an effort to determine the behavior of a woodframe structure under various levels of seismic intensity and to determine the influence of various structural configurations on the performance of the structure. The results and conclusions developed from these seismic tests are presented in this chapter. This chapter begins with a discussion of the shake table fidelity during the seismic tests. A description of the seismic test results is provided along with descriptions of the selected test results incorporated in various appendices. Selected comparative seismic responses are then discussed. This chapter concludes with a summary of the key results obtained from the seismic tests.
10.1 Shake Table Fidelity

One important aspect of shake table testing is the capacity of accurately reproducing the reference or input motion. The shake table fidelity can be evaluated by comparing absolute acceleration response spectra for the recorded table acceleration feedback with the input table acceleration. Appendix H shows absolute acceleration response spectra at 5% damping of the feedback acceleration and the reference acceleration for each of the seismic tests performed on the test structure.

For the low amplitude tests of each phase, the spectral accelerations of feedback signals are slightly higher than the reference spectral accelerations in the period range of the structure (0.15 sec to 0.32 sec). This was most likely a result of an inability of the hydraulic system to accurately deliver such small motions. The feedback response matched the reference response very well for the higher amplitude ground motions over a large range of structure periods. The feedback response for the Level 5 tests using the near fault ground motion was lower than the reference response in the higher period ranges as a result of hydraulic limitations. However, in the period range of the structure, the feedback and reference responses matched relatively well.

10.2 Description of Test Results

From the extensive data that was recorded during the seismic tests, the key seismic results have been extracted and are presented in various appendices. There are five separate appendices for the selected seismic test results. The five types of seismic results presented include: i) visual damage observation (Appendix I), ii) displacement and acceleration measurements (Appendix J), iii) base shear force-displacement
hysteresis loops (Appendix K), iv) anchor bolt force measurements (Appendix L), v) sill plate and holdown stud uplift measurements (Appendix M), and vi) sill plate slippage measurements (Appendix N). For each type of seismic results, the corresponding appendix is described and general observations regarding peak recorded values and other trends are noted.

10.2.1 Visual Damage Observations

Visual damage observations were made following each seismic test. Selected visual damage observations are presented in Appendix I for each phase of seismic testing. These damage observations show that the damage to the test structure was relatively small considering the severity of the ground motions. There was numerous occurrences of cracking and splitting of sill plates during Phase 5. Once the sill plates cracked and split during Phase 5, there was minimal damage to the sill plates in subsequent phases. The nailing of the CS16 inter-story holdown straps and HTT22 holdowns caused extensive splitting of the holdown studs. Several of the splits originated from the nailing during construction and extended during the seismic shaking. Another damage observation common to several of the phases was the cracking of the OSB wall sheathing at the upper corners of first story door openings. Crushing of the foundation grout was common at the door openings for many of the phases of testing. Although visual observation was not possible, the majority of the damage to the lateral load resisting system was crushing of the wood fibers around the sheathing nails. This damage was evident by the decrease in the fundament frequency of the test structure with each level as was discussed previously in Chapter 7.
10.2.2 **Displacement and Acceleration Measurements**

Relative displacement and absolute acceleration time-histories were plotted for several key locations in the test structure. Appendix J shows relative displacement time-histories in the North-South direction for the east and west walls at the floor level, eave level, and roof ridge. In addition, this appendix presents the absolute acceleration time-histories in the North-South direction for the east and west walls at the floor level, eave level, and roof ridge.

The minimum relative displacement at the roof ridge of all seismic tests was 0.04 inch occurring during the Level 1 test of Phase 5. The maximum relative displacement at the roof ridge of all seismic tests was 4.37 inches occurring during Level 5 test of Phase 9. The minimum absolute acceleration at the roof ridge of all seismic tests was 0.08 g occurring during Level 1 test of Phase 5. The maximum absolute acceleration at the roof ridge of all seismic tests was 1.62 g occurring during the Level 5 test of Phase 9. The Level 5 test of Phase 9 was the only seismic test in which a noticeable residual displacement (0.27 inch at the roof) was recorded.

10.2.3 **Base Shear Force-Displacement Hysteresis Loops**

Global base shear force-displacement hysteresis loops developed for each seismic test are presented in Appendix K. The base shear was computed by adding the first and second story inertia forces. The second story inertia force was determined by multiplying the weight of upper half of the second story walls and roof (10.8 kips) by the roof acceleration at the centerline of the structure. The first story inertia force was determined by multiplying the weight of the lower half of the second story walls, the upper half of the first story walls, the floor diaphragm, and the second story interior
partition walls (13.8 kips) by the floor acceleration at the centerline of the structure. Note that this base shear is the total force induced in the foundation of the test structure including the nonlinear restoring force and the viscous damping force. The relative displacement at the roof eaves at the centerline of the structure was used for the base shear force-displacement hysteresis loops. The base shear force ranged from 1.6 kips during the Level 1 test of Phase 7 to 34.7 kips during the Level 5R test of Phase 10.

### 10.2.4 Anchor Bolt Force Measurements

The peak tensile anchor bolt forces developed by each of the shear transfer and holdown anchor bolts during each seismic test are shown as three-dimensional bar graphs in Appendix L. A bar scaled to the peak anchor bolt force (in pounds) is shown at the location of each anchor bolt in the structure. Only the peak tensile loads in the anchor bolt are shown since compressive loads typically are not developed in anchor bolts.

The peak anchor bolt forces ranged from nearly zero during the Level 1 test of Phase 5 to 9520 pounds during the Level 5 test of Phase 9. The peak anchor bolt forces were very low during all Level 1 and Level 2 seismic tests since the gravity loads acting on the shearwalls exceeded the uplift or overturning loads induced by the seismic shaking. For the higher amplitude levels, the peak anchor bolt forces were considerably higher in the holdown anchor bolts as compared with the shear transfer anchor bolts. This is to be expected since the holdown devices at the end of the shearwall segments resist the majority of the overturning. For test Phases 5 to 9, the peak anchor bolt forces were relatively low in the north and south walls as compared
to the peak anchor bolt forces in the east and west shearwalls. However, the peak anchor bolt forces are more evenly distributed to all four walls during Phase 10. This phenomenon will be discussed in more detail in a later section. The peak anchor bolt forces were relatively low in the interior-bearing wall since shear-resisting panels were not installed on this wall.

10.2.5 Sill Plate and Holdown Stud Uplift Measurements

The uplift of the sill plates and holdown studs at shearwall segment ends were measured for all seismic tests. On the first story, the sill plate uplift was the vertical deflection of the sill plate relative to the steel base. The holdown stud uplift on the first story was the vertical deflection of the holdown stud (stud with HTT22 holdown attached) relative to the sill plate. On the second story, the sill plate uplift was the vertical deflection of the bottom wall plate relative to the floor rim joists. The holdown stud uplift on the second story was the vertical deflection of the holdown stud relative to the bottom plate. The total uplift (relative to the base or floor rim joists) is the sum of the sill plate uplift and holdown stud uplift. Bar graphs showing the peak sill plate uplift and the total uplift are shown in Appendix M for each seismic test.

The total uplift ranges from nearly zero during the Level 1 test of Phase 5 to 0.85 inch during the Level 4 test of Phase 7. This total uplift can be divided into the peak sill plate uplift and peak holdown stud uplift. The peak sill plate uplift ranged from nearly zero to 0.25 inch during the Level 4 test of Phase 7. Occurring at a different location, the peak holdown stud uplift for all seismic tests was 0.65 inch occurring during this same test. The peak sill plate and holdown stud uplift were relatively small (typically less than 0.04 inch) on the second story for nearly all of the
seismic tests. This uplift was very small on the second story because the uplift was resisted by the CS16 inter-story straps, nailing of the bottom plate, and overlapping of the wall sheathing. The uplift on the first story was much larger because only the anchor bolts resisted the uplift. The uplift was small on the walls perpendicular (typically less than 0.06 inch) to the direction of shaking (north and south walls).

10.2.6 Sill Plate Slippage Measurements

The slippage of the first and second story sill plates was also measured during the seismic tests. The slippage of the first story sill plates was measured relative to the steel base and the slippage of the second story sill plates was measured relative to the floor sheathing. Since the sill plates could slip in either direction parallel to the sill plate, positive and negative sill plate slippage measurements were made. Appendix N shows the peak sill plate slippage values as horizontal bar graphs measured for each of the exterior shearwalls in the test structure during each seismic test.

The peak slippage ranged from nearly zero for the lower level tests to 0.22 inch in the north direction and 0.21 inch in the south direction for the Level 5R test of Phase 10. For the test phases in which the sill plate was continuous with no openings in the shearwalls, the sill plate slippage for both sill plate segments was approximately equal. However, if the sill plate was discontinuous with a door opening, the sill plate slippage for each sill plate segment was independent of one another. Unlike the sill plate uplift, there was a recognizable sill plate slippage (between 0.001 inch and 0.14 inch) on the second story sill plates. For both stories, the sill plate slippage of the north and south sill plates (slippage perpendicular to direction of shaking) was very small.
10.3 Selected Comparative Seismic Responses

Using the seismic test results that are presented in the appendices and described in the previous section, the seismic responses can be compared for different phases to determine the effects of different structural configurations on the seismic performance of the test structure. This section begins with a discussion of capacity spectra developed for each test phase. These capacity spectra present a general overview of the seismic performance of the various structural configurations. The effect of repeated moderate ground motions is also discussed. The in-plane response of the floor diaphragm and first story shearwalls are compared in order to classify the diaphragm flexibility during dynamic response. The effect of symmetrical openings on the seismic response of the structure is then discussed followed by a discussion on the effect of non-symmetrical wall openings. As part of a minor addition to test Phase 6, the effect of the waste wall sheathing above and below window openings are compared. Comparisons are also made regarding the performance of engineered, perforated shearwall, and conventional constructions. The final comparative study involves a discussion on the influence of wall finish materials on the structure response. This effect of wall finish materials proved to be the most important finding of the seismic tests.

10.3.1 Capacity Spectra

Capacity spectra were developed for each phase of seismic testing to compare the global response of the structure for each test phase. A plot of maximum base shear versus the corresponding peak roof relative displacement measured for each test level was generated for each test phase as shown in Figure 10.1. Note that this base shear is
the total force induced in the foundation of the test structure including the nonlinear restoring force and the viscous damping force.

![Graph showing capacity spectra for phases of seismic testing](image)

Figure 10.1: Capacity Spectra for Phases of Seismic Testing

Several relevant conclusions can be drawn from these capacity spectra. A comparison of the capacity spectra for test Phases 5 and 6 shows that the fully sheathed test structure is stiffer than the structure with symmetrical openings. The peak roof absolute acceleration for the structure with symmetrical openings is slightly higher than the fully sheathed structure. A comparison of global response of the structure using perforated shearwall construction (Phase 7) with the structure using conventional construction (Phase 8) shows that the perforated shearwall design did not have a significant influence on the overall behavior of the structure. The test structure
with non-symmetrical openings (Phase 9) showed a similar response to Phase 7 and 8 up to Level 4. Since the test structure of Phase 9 was tested under the Level 5 near fault ground motion, there was significant inelastic behavior in the structure. The test structure of Phase 10 with wall finish materials exhibited a nearly linear response with a stiffness higher than the fully sheathed test structure of Phase 5. This suggests that wall finish materials had a profound effect on the seismic response of the test structure.

10.3.2 Effect of Repeated Moderate Ground Motions

As part of the testing protocol, one seismic test for each phase was repeated when the peak inter-story drift at any wall line fell between 0.5% and 1%. For Phases 5, 6, and 10, the test Level 4 was repeated. Seismic test Level 3 was repeated for Phases 7, 8, and 9. The main objective of repeating one of the seismic tests was to determine the seismic performance of the structure after it was subjected to two moderate earthquakes without any repairs conducted after the first seismic event. Table 10.1 presents the peak response parameters following both successive seismic tests in each test phase.

A comparison of the results for each phase shows that there is a significant change in the seismic performance of the structure after the repeated ground motions. The fundamental frequency decreased by an average of 3% after the repeated seismic test. The peak roof absolute acceleration and the peak roof relative displacement increased an average of 13% and 20%, respectively. These substantial increases of absolute acceleration and relative displacement could cause problems for non-
<table>
<thead>
<tr>
<th>Test Phase</th>
<th>Test Level</th>
<th>Fundamental Frequency (Hz)</th>
<th>Peak Roof Absolute Acceleration (g)</th>
<th>Peak Roof Relative Displacement (in)</th>
<th>Peak Anchor Bolt Force (lbs)</th>
<th>Peak Sill Plate + Holdown Stud Uplift (in)</th>
<th>Peak Sill Plate Slippage (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 4</td>
<td>4</td>
<td>5.13</td>
<td>0.90</td>
<td>0.85</td>
<td>1979</td>
<td>0.126</td>
<td>0.061</td>
</tr>
<tr>
<td>5 4R</td>
<td>4R</td>
<td>4.98</td>
<td>1.06</td>
<td>1.15</td>
<td>2441</td>
<td>0.144</td>
<td>0.099</td>
</tr>
<tr>
<td>6 4</td>
<td>4</td>
<td>3.66</td>
<td>1.10</td>
<td>1.71</td>
<td>3479</td>
<td>0.264</td>
<td>0.110</td>
</tr>
<tr>
<td>6 4R</td>
<td>4R</td>
<td>3.71</td>
<td>1.25</td>
<td>2.14</td>
<td>4318</td>
<td>0.295</td>
<td>0.125</td>
</tr>
<tr>
<td>7 3</td>
<td>3</td>
<td>3.66</td>
<td>0.83</td>
<td>1.45</td>
<td>3414</td>
<td>0.271</td>
<td>0.100</td>
</tr>
<tr>
<td>7 3R</td>
<td>3R</td>
<td>3.56</td>
<td>0.87</td>
<td>1.53</td>
<td>3568</td>
<td>0.297</td>
<td>0.101</td>
</tr>
<tr>
<td>8 3</td>
<td>3</td>
<td>3.91</td>
<td>0.69</td>
<td>1.27</td>
<td>1859</td>
<td>0.125</td>
<td>0.128</td>
</tr>
<tr>
<td>8 3R</td>
<td>3R</td>
<td>3.81</td>
<td>0.79</td>
<td>1.45</td>
<td>2049</td>
<td>0.159</td>
<td>0.131</td>
</tr>
<tr>
<td>9 3</td>
<td>3</td>
<td>3.66</td>
<td>0.75</td>
<td>1.41</td>
<td>4723</td>
<td>0.213</td>
<td>0.087</td>
</tr>
<tr>
<td>9 3R</td>
<td>3R</td>
<td>3.42</td>
<td>0.86</td>
<td>1.73</td>
<td>5982</td>
<td>0.234</td>
<td>0.103</td>
</tr>
<tr>
<td>10 4</td>
<td>4</td>
<td>5.71</td>
<td>0.92</td>
<td>0.61</td>
<td>1864</td>
<td>0.104</td>
<td>0.138</td>
</tr>
<tr>
<td>10 4R</td>
<td>4R</td>
<td>5.47</td>
<td>1.06</td>
<td>0.71</td>
<td>2386</td>
<td>0.117</td>
<td>0.137</td>
</tr>
</tbody>
</table>

Mean Percentage of Change: -3% +13% +20% +19% +14% +16%

structural finish materials if a structure was not repaired following a moderate ground motion. The peak anchor bolt forces increased by an average of 19% following the repeated seismic test as a result of increased overturning in the structure. The peak total uplift also increased by an average of 14% due to the increased overturning. In addition, the peak sill plate slippage increased by an average of 16% following the repeated seismic test. The results of these repeated moderate ground motions suggest that if the lateral load resisting system is not repaired following a moderate ground motion, the structural response due to subsequent moderate ground motions could deteriorate substantially.
10.3.3  Response of Floor Diaphragm and Shearwalls

Using the relative displacement measurements of the first story east and west shearwalls and the relative displacement measurements of the floor diaphragm, the deflected shape of the floor diaphragm could be determined. The flexibility of the floor diaphragm can be classified as either rigid or flexible depending on the deflected shape of the floor diaphragm. As discussed previously in Chapter 7, UBC Section 1630.6 classifies a diaphragm as rigid if the diaphragm deflection is less than two times the average shearwall relative displacement (ICBO, 1997). Otherwise, the diaphragm is classified as flexible. For each of the seismic tests, the flexibility of the diaphragm was classified according to UBC Section 1630.6 as shown in Table 10.2.

The maximum floor diaphragm to shearwall deflection ratio was 0.71 occurring during Seismic Test 8.S.1. As a result, the floor diaphragm would be classified as rigid for all of the seismic tests according to UBC Section 1630.6. This classification has a significant effect on the design of a woodframe structure. For a rigid diaphragm, a torsional moment is developed when the centroid of the applied force (that acts through the center of mass) does not coincide with the center of rigidity of the resisting shearwalls. The center of mass does not coincide with the center of rigidity of the test structure for Phase 9 and Phase 10. As a result, more lateral load would be induced into the shearwalls.
<table>
<thead>
<tr>
<th>Seismic Test</th>
<th>Maximum Diaphragm Deflection, $\Delta_{\text{global}}$ (in)</th>
<th>Maximum Shearwall Deflection, $\Delta_{\text{shearwall}}$ (in)</th>
<th>Diaphragm to Shearwall Deflection Ratio, $\frac{\Delta_{\text{global}}}{\Delta_{\text{shearwall}}}$</th>
<th>Diaphragm Classification According to UBC 1630.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.S.1</td>
<td>0.003</td>
<td>0.029</td>
<td>0.11</td>
<td>Rigid</td>
</tr>
<tr>
<td>5.S.2</td>
<td>0.018</td>
<td>0.073</td>
<td>0.25</td>
<td>Rigid</td>
</tr>
<tr>
<td>5.S.3</td>
<td>0.027</td>
<td>0.204</td>
<td>0.13</td>
<td>Rigid</td>
</tr>
<tr>
<td>5.S.4</td>
<td>0.053</td>
<td>0.483</td>
<td>0.11</td>
<td>Rigid</td>
</tr>
<tr>
<td>5.S.4R</td>
<td>0.056</td>
<td>0.651</td>
<td>0.09</td>
<td>Rigid</td>
</tr>
<tr>
<td>6.S.1</td>
<td>0.018</td>
<td>0.037</td>
<td>0.48</td>
<td>Rigid</td>
</tr>
<tr>
<td>6.S.2</td>
<td>0.044</td>
<td>0.218</td>
<td>0.20</td>
<td>Rigid</td>
</tr>
<tr>
<td>6.S.3</td>
<td>0.058</td>
<td>0.461</td>
<td>0.13</td>
<td>Rigid</td>
</tr>
<tr>
<td>6.S.4</td>
<td>0.124</td>
<td>0.885</td>
<td>0.14</td>
<td>Rigid</td>
</tr>
<tr>
<td>6.S.4R</td>
<td>0.158</td>
<td>1.091</td>
<td>0.14</td>
<td>Rigid</td>
</tr>
<tr>
<td>6A.S.3</td>
<td>0.125</td>
<td>0.845</td>
<td>0.15</td>
<td>Rigid</td>
</tr>
<tr>
<td>7.S.1</td>
<td>0.022</td>
<td>0.054</td>
<td>0.40</td>
<td>Rigid</td>
</tr>
<tr>
<td>7.S.2</td>
<td>0.079</td>
<td>0.434</td>
<td>0.18</td>
<td>Rigid</td>
</tr>
<tr>
<td>7.S.3</td>
<td>0.109</td>
<td>0.827</td>
<td>0.13</td>
<td>Rigid</td>
</tr>
<tr>
<td>7.S.3R</td>
<td>0.116</td>
<td>0.861</td>
<td>0.13</td>
<td>Rigid</td>
</tr>
<tr>
<td>7.S.4</td>
<td>0.171</td>
<td>1.325</td>
<td>0.13</td>
<td>Rigid</td>
</tr>
<tr>
<td>7A.S.3</td>
<td>0.122</td>
<td>0.996</td>
<td>0.12</td>
<td>Rigid</td>
</tr>
<tr>
<td>7A.S.4</td>
<td>0.172</td>
<td>1.369</td>
<td>0.13</td>
<td>Rigid</td>
</tr>
<tr>
<td>8.S.1</td>
<td>0.020</td>
<td>0.029</td>
<td>0.71</td>
<td>Rigid</td>
</tr>
<tr>
<td>8.S.2</td>
<td>0.064</td>
<td>0.249</td>
<td>0.26</td>
<td>Rigid</td>
</tr>
<tr>
<td>8.S.3</td>
<td>0.096</td>
<td>0.639</td>
<td>0.15</td>
<td>Rigid</td>
</tr>
<tr>
<td>8.S.3R</td>
<td>0.103</td>
<td>0.725</td>
<td>0.14</td>
<td>Rigid</td>
</tr>
<tr>
<td>8.S.4</td>
<td>0.171</td>
<td>1.405</td>
<td>0.12</td>
<td>Rigid</td>
</tr>
<tr>
<td>9.S.1</td>
<td>0.022</td>
<td>0.057</td>
<td>0.39</td>
<td>Rigid</td>
</tr>
<tr>
<td>9.S.2</td>
<td>0.080</td>
<td>0.330</td>
<td>0.24</td>
<td>Rigid</td>
</tr>
<tr>
<td>9.S.3</td>
<td>0.104</td>
<td>0.740</td>
<td>0.14</td>
<td>Rigid</td>
</tr>
<tr>
<td>9.S.3R</td>
<td>0.126</td>
<td>0.912</td>
<td>0.14</td>
<td>Rigid</td>
</tr>
<tr>
<td>9.S.4</td>
<td>0.177</td>
<td>1.463</td>
<td>0.12</td>
<td>Rigid</td>
</tr>
<tr>
<td>9.S.5</td>
<td>0.468</td>
<td>2.388</td>
<td>0.20</td>
<td>Rigid</td>
</tr>
<tr>
<td>10.S.1</td>
<td>0.013</td>
<td>0.026</td>
<td>0.51</td>
<td>Rigid</td>
</tr>
<tr>
<td>10.S.2</td>
<td>0.033</td>
<td>0.089</td>
<td>0.37</td>
<td>Rigid</td>
</tr>
<tr>
<td>10.S.3</td>
<td>0.073</td>
<td>0.192</td>
<td>0.38</td>
<td>Rigid</td>
</tr>
<tr>
<td>10.S.4</td>
<td>0.042</td>
<td>0.490</td>
<td>0.09</td>
<td>Rigid</td>
</tr>
<tr>
<td>10.S.4R</td>
<td>0.058</td>
<td>0.527</td>
<td>0.11</td>
<td>Rigid</td>
</tr>
<tr>
<td>10.S.5</td>
<td>0.071</td>
<td>0.702</td>
<td>0.10</td>
<td>Rigid</td>
</tr>
<tr>
<td>10.S.5R</td>
<td>0.087</td>
<td>0.909</td>
<td>0.10</td>
<td>Rigid</td>
</tr>
</tbody>
</table>
10.3.4 Effect of Symmetrical Wall Openings (Phases 5 and 6)

One of the objectives of the seismic testing was to determine the effect of symmetrical openings on the seismic response of the structure. The structural response of the fully sheathed test structure of Phase 5 and the test structure with symmetrical openings of Phase 6 are compared in terms of relative displacement time-histories, absolute acceleration time-histories, base shear force-displacement hysteresis loops, energy absorbed time-histories, peak anchor bolt forces, first story sill plate and holdown stud uplift, and first story sill plate slippage.

The presence of symmetrical door and window openings in the east and west walls caused a significant increase in the relative displacement of the test structure. Figure 10.2 shows the Level 4 roof relative displacement time-histories for the fully sheathed test structure and test structure with symmetrical openings. The peak relative roof displacement increased by a factor of two when the symmetrical openings are introduced. This increase in lateral displacement was most likely due to a combination of a decrease in lateral stiffness and an increase in overturning. The peak absolute roof

![Figure 10.2: Comparison of Level 4 Roof Relative Displacement Time-Histories for Test Phase 5 and Test Phase 6](image-url)
acceleration also increased by about 25% with the presence of symmetrical window and door openings as shown in Figure 10.3.

![Graph showing comparison of Level 4 Roof Absolute Acceleration Time-Histories for Test Phase 5 and Test Phase 6](image)

**Figure 10.3: Comparison of Level 4 Roof Absolute Acceleration Time-Histories for Test Phase 5 and Test Phase 6**

When the symmetrical door and window openings were introduced during test Phase 6, there was a significant decrease in lateral stiffness of the structure. Figure 10.4 shows a comparison of the Level 4 base shear force-displacement hysteresis loops for the fully sheathed test structure and the test structure with symmetrical openings. Along with the decrease in lateral stiffness, there was a moderate increase in the base shear force for the test structure with symmetrical openings. The shape of these hysteresis loops can be compared by examining the energy absorbed time-histories for each of the two test structures. Energy absorbed time-histories were developed for the two test structures by numerically integrating the base shear force-displacement hysteresis loops. Figure 10.5 shows a comparison of the energy absorbed time-histories for the fully sheathed test structure and the test structure with symmetrical openings. The total energy absorbed nearly doubles once the symmetrical openings were introduced. As a result of the decrease in shearwall length with the
symmetrical openings, each shearwall segment would have a higher lateral load per unit length. The combination of this increased load and the increased deformation due to the reduction in lateral stiffness, the nail deformation was greater when the symmetrical openings are introduced causing the increase in absorbed energy.

![Figure 10.4: Comparison of Level 4 Base Shear Force-Displacement Hysteresis Loops for Test Phase 5 and Test Phase 6](image)

![Figure 10.5: Comparison of Level 4 Energy Absorbed Time-Histories for Test Phase 5 and Test Phase 6](image)

Since additional HTT22 holdown devices were installed at the end of each shearwall segment adjacent to the door openings for Phase 6, there was a redistribution of the anchor bolt forces from Phase 5 to Phase 6. Figure 10.6 shows the Level 4 peak anchor bolt forces for the fully sheathed test structure and the test
structure with symmetrical openings. Since the shearwall segments are 16 feet long in
the fully sheathed test structure, the overturning is small as shown by the low holdown
anchor bolt forces at the ends of the east and west shearwalls. However, the
symmetrical door openings of Phase 5 create two shearwall segments 6.5 feet long,
which are subjected to larger overturning moments. As a result, the holdown anchor
bolt forces at the shearwall segment ends are significantly higher in Phase 6 than in
Phase 5. There was also a minor increase in the anchor bolt forces in the north and
south walls with the symmetrical door and window openings of Phase 6.

The increased overturning in the east and west shearwalls with the symmetrical
door openings also caused an increase in the sill plate and holdown stud uplift. Figure
10.7 shows for the Level 4 test the first story peak sill plate and holdown stud uplift
for the fully sheathed test structure and the test structure with symmetrical openings.
This increase in sill plate and holdown stud uplift coincides with the increase in peak
anchor bolt forces caused by the increased overturning in the shorter shearwall
segments.

The symmetrical door openings also had a significant effect on the sill plate
slippage during the seismic tests of Phase 6. The first story peak sill plate slippage was
approximately constant along the continuous sill plate of the fully sheathed test
structure as shown in Figure 10.8. However, once the door openings of Phase 6
divided the sill plate into two separate segments, each sill plate segment slipped by a
different amount. The sill plate slippage for the test structure with symmetrical
openings was larger than the sill plate slippage of the fully sheathed structure. For the
sill plates with low slippage, a particular anchor bolt may have been closer to one side
Phase 5 – Fully Sheathed

Phase 6 – Symmetrical Openings

Figure 10.6: Comparison of Level 4 Peak Anchor Bolt Forces for Phase 5 and Phase 6
Phase 5 – Fully Sheathed

Phase 6 – Symmetrical Openings

Figure 10.7: Comparison of Level 4 First Story Peak Sill Plate and Holdown Stud Uplift for Test Phase 5 and Test Phase 6

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Phase 5 – Fully Sheathed

Phase 6 – Symmetrical Openings

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.

Figure 10.8: Comparison of Level 4 First Story Peak Sill Plate Slippage for Test Phase 5 and Test Phase 6
of an oversized hole and another anchor bolt may have been closer to the other side of an oversized hole. For the sill plates with high slippage, however, both anchor bolts may have been closer to the same side of oversized holes. Figure 10.9 shows a comparison of probable anchor bolt locations for low sill plate slippage and high sill plate slippage conditions.

![Figure 10.9: Comparison of Low and High Sill Plate Slippage Conditions](image)

10.3.5 Effect of Non-Symmetrical Wall Openings (Phases 6 and 9)

Another objective of the seismic testing was to determine the effect of non-symmetrical openings on the seismic response of the structure. The test structure with symmetrical openings of Phase 6 and the structure with non-symmetrical openings of Phase 9 are compared in terms of relative displacement time-histories, absolute acceleration time-histories, base shear force-displacement hysteresis loops, energy absorbed time-histories, peak anchor bolt forces, first story sill plate and holdown stud uplift, and first story sill plate slippage.

The large garage door opening on the first story east wall caused a significant increase in the relative displacement. Figure 10.10 shows a comparison of the Level 4 relative displacement time-histories of the first story east wall for the structure with symmetrical openings and the structure with non-symmetrical openings. This increase
in relative displacement is due to the reduction in lateral stiffness with the large opening as well as an increase in overturning. There was also torsional deformation of the structure with non-symmetrical opening during Phase 9. Figure 10.11 shows a comparison of relative displacement time-histories for the first story east and west shearwalls for Level 4 of Phase 9. The torsional behavior of the structure can be seen by the higher relative displacement of the first story east shearwall. It is to be expected that the structure deformed torsionally since the diaphragm was nearly rigid as discussed previously.

![Figure 10.10: Comparison of Level 4 First Story East Wall Relative Displacement Time-Histories for Test Phase 6 and Test Phase 9](image)

![Figure 10.11: Comparison of Level 4 First Story East and West Wall Relative Displacement for Test Phase 9](image)
Although the structure deformed in translation and torsion, the effect of non-symmetrical openings is not shown in absolute acceleration time histories. Figure 10.12 compares the Level 4 roof absolute acceleration time-histories for the structure with symmetrical openings and the structure with non-symmetrical openings. There is only a marginal increase in the roof absolute acceleration with the non-symmetrical openings of Phase 9. In addition, the absolute acceleration response for the east and west walls are approximately equal for the non-symmetric structure of Phase 9.

The reduction in lateral stiffness of the test structure incorporating non-symmetrical openings can be seen in the Level 4 base shear force-displacement response as shown in Figure 10.13. The base shear force remained essentially the same when the large opening was introduced to the test structure during test Phase 9. However, the lateral load per unit length of shearwall increased in the east wall of the Phase 9 test structure. With this increased load per unit length and increased deformation of the shearwall, there was a moderate increase in the absorbed energy in the structure incorporating the non-symmetrical openings. Figure 10.14 shows a
comparison of the Level 4 energy absorbed time-histories for the test structure with symmetrical openings and the test structure with non-symmetrical openings. This increase in absorbed energy in the structure with non-symmetrical openings probably resulted from increased nail deformation.

![Base Shear Force-Displacement Hysteresis Loops for Test Phase 6 and Test Phase 9](image1)

**Figure 10.13:** Comparison of Level 4 Base Shear Force-Displacement Hysteresis Loops for Test Phase 6 and Test Phase 9

![Energy Absorbed Time-Histories for Test Phase 6 and Test Phase 9](image2)

**Figure 10.14:** Comparison of Level 4 Energy Absorbed Time-Histories for Test Phase 6 and Test Phase 9

As a result of increased overturning in the narrow shearwall segments of the first story east wall of Phase 9, the peak anchor bolt forces were significantly higher. Figure 10.15 shows the Level 4 peak anchor bolt forces for Phase 6 and Phase 9. For
example, the peak anchor bolt force at the holdown at the south end of the east shearwall increased by a factor of 2.6 due to the narrow shearwall segments. In addition, the anchor bolts in the first story west wall showed an increase in force in Phase 9 compared to Phase 6. The peak anchor bolt forces in the north and south walls increased slightly during Phase 9 possibly as a result of the torsional behavior of the test structure.

The increased overturning in the first story east shearwall segments shown by the increased peak anchor bolt forces can also be seen in sill plate and holdown stud uplift. Figure 10.16 shows a comparison of the first story sill plate and holdown stud uplift for the structure with symmetrical openings and the structure with non-symmetrical openings. Similar to the peak anchor bolt forces, the sill plate and holdown stud uplift increased significantly with the presence of the garage door opening. In addition, the peak sill plate and holdown stud uplift in the west wall increased slightly during Phase 9. The sill plate uplift in the first story north and south walls increased slightly with the non-symmetrical openings. The sill plate and holdown stud uplift for the second story was very small for both Phase 6 and Phase 9 with very little difference between the two configurations.

The first story sill plate slippage for the structure with non-symmetrical openings was higher in some locations and lower in others. Figure 10.17 shows a comparison of the Level 4 first story sill plate slippage for the structure with symmetrical openings and the structure with non-symmetrical openings. The first story east wall sill plate slippage decreased when the large garage door opening was
Phase 6 – Symmetrical Openings

Phase 9 – Non-Symmetrical Openings

Figure 10.15: Comparison of Level 4 Peak Anchor Bolt Forces for Test Phase 6 and Test Phase 9
Phase 6 – Symmetrical Openings

Phase 9 – Non-Symmetrical Openings

Figure 10.16: Comparison of Level 4 First Story Peak Sill Plate and Holdown Stud Uplift for Test Phase 6 and Test Phase 9

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Phase 6 – Symmetrical Openings

Phase 9 – Non-Symmetrical Openings

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.

Figure 10.17: Comparison of Level 4 First Story Peak Sill Plate Slippage for Test Phase 6 and Test Phase 9
introduced during Phase 9. As discussed previously, the first story sill plate slippage is primarily a function of the oversized holes for the anchor bolts. Since a portion of the first story east sill plate was removed for the garage door opening and two additional shear transfer anchor bolts were installed in the remaining sill plate for Phase 9, the location of the anchor bolts in the oversized holes was changed. Since the sill plate slippage was less for Phase 9, the anchor bolts in the sill plate were most likely located at different sides of the holes. The sill plate slippage for the first story west wall did not change with the non-symmetrical opening configuration since the sill plate was unaltered between the two tests. The sill plate slippage perpendicular to the direction of shaking in the north and south walls increased slightly during Phase 9 possibly due to the torsional response of the structure. The slippage for the second story sill plates remained essentially the same for the structures with symmetrical and non-symmetrical openings.

10.3.6 Effect of Waste Wall Sheathing (Phases 6 and 6A)

Since shearwalls are typically designed neglecting the influence of the waste wall sheathing above and below window openings, test Phase 6A was established to determine the effect of this sheathing. Since the seismic tests of Phase 6 included the waste wall sheathing above and below the second story window openings, the waste wall sheathing on the east and west walls was removed for Phase 6A as was shown in Figure 4.14. Recall that only the Level 3 seismic test (with frequency and damping evaluation tests performed before and after) was conducted for Phase 6A. The structural response of Phase 6 and Phase 6A are compared in terms of fundamental
frequency, structural mode shapes, relative displacement time-histories, absolute acceleration time-histories, base shear force-displacement hysteresis loops, energy absorbed time-histories, second story sill plate and holdown stud uplift, and second story sill plate slippage.

From the frequency evaluations tests, the fundamental frequency decreased from 3.71 Hz to 3.27 Hz when the waste wall sheathing was removed for Phase 6A. Using a SDOF assumption for the structure (described previously in Chapter 8), this decrease in fundamental frequency corresponds to a 22% reduction in lateral stiffness of the structure. The structural mode shape changed with the removal of the waste wall sheathing during Phase 6A as shown in Figure 10.18. The modal values at the first story east and west wall were 0.52 and 0.51, respectively, for the structure with the waste wall sheathing. Once the waste wall sheathing was removed, these modal values decreased to 0.46 and 0.47, respectively suggesting some reduction in the second story lateral stiffness. A comparison of the Level 3 relative roof displacement for the structure with waste wall sheathing and the structure without waste wall sheathing shows a significant increase as shown by Figure 10.19. This increase in relative displacement of more than a factor of two suggests that the waste wall sheathing has a significant effect on the performance of the structure. A comparison of the first story relative displacement shows that the structure without the waste sheathing had an increased relative displacement as well. One would expect only the second story inter-story relative displacement to increase since only the second story waste wall sheathing was removed. However, the removal of the waste wall sheathing changed the fundamental frequency of the structure, potentially causing an increase in the input
seismic energy. A comparison of the Level 3 roof absolute acceleration time histories for the two structural configurations also shows an increased seismic response with no waste wall sheathing as shown by Figure 10.20.

**With Waste Wall Sheathing**

With Waste Wall Sheathing

**Without Waste Wall Sheathing**

Without Waste Wall Sheathing
A comparison of the Level 4 base shear force-displacement response of the test structure with waste wall sheathing and the test structure without waste wall sheathing shows a reduction in lateral stiffness and an increase in nonlinear response as shown in Figure 10.21. In addition, there was an increase in base shear for the structure without waste wall sheathing. The increase in nonlinear response for the structure without waste wall sheathing resulted from an increased lateral load per unit length of
shearwall and an increased lateral deformation causing more nonlinear nail deformations. This increase in nonlinear response can be seen in a comparison of the test Level 4 absorbed energy time histories for the test structure with waste wall sheathing and the test structure without waste wall sheathing as shown in Figure 10.22.

![Comparison of Level 4 Base Shear Force-Displacement Hysteresis Loops for Test Phase 6 and Test Phase 6A](image)

**Figure 10.21:** Comparison of Level 4 Base Shear Force-Displacement Hysteresis Loops for Test Phase 6 and Test Phase 6A

![Comparison of Level 4 Energy Absorbed Time-Histories for Test Phase 6 and Test Phase 6A](image)

**Figure 10.22:** Comparison of Level 4 Energy Absorbed Time-Histories for Test Phase 6 and Test Phase 6A

The first story sill plate uplift remained essentially the same with the removal of the waste wall sheathing since only the second story waste wall sheathing was removed. However, the second story sill plate and holdown uplift was higher for Phase
6A without the waste wall sheathing as shown in Figure 10.23. Since the removal of the waste wall sheathing above and below the window openings created 3-foot long shearwall segments, the overturning in the shearwalls was significantly higher than the 16-foot long shearwall of Phase 6. As a result of the increased overturning, the sill plate and holdown stud uplift was increased. If force measurements could have been made on the CS16 inter-story holdown straps, an increase in the holdown forces would probably have been seen as well.

In addition to an increase in the sill plate and holdown stud uplift, there was also an increase in the second story sill plate slippage. Figure 10.24 shows the Level 3 second story sill plate slippage for the structure with waste wall sheathing and the structure without waste wall sheathing. Since the second story sill plate was continuous over the 16-foot width of the building, the removal of the waste wall sheathing should probably not have a significant effect on the sill plate slippage. However, as discussed previously, the changed in fundamental frequency of the structure with the removal of waste wall sheathing could have caused an increased input energy to the structure. As a result, the lateral force in the second story shearwalls could have been higher leading to the increased sill plate slippage.
Phase 6 – With Waste Wall Sheathing

Phase 6A – Without Waste Wall Sheathing

2nd Story Uplift Configuration

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.

Figure 10.23: Comparison of Level 3 Second Story Peak Sill Plate and Holdown Stud Uplift for Test Phase 6 and Test Phase 6A
Phase 6 – With Waste Wall Sheathing

Phase 6A – Without Waste Wall Sheathing

Figure 10.24: Comparison of Level 3 Second Story Peak Sill Plate Slippage for Test Phase 6 and Test Phase 6A

Note: Sill plate slippage measured relative to floor sheathing on 2nd story. All values are in inches.
10.3.7 Performance of Perforated Shearwalls (Phases 6 & 7)

The main objective of test Phase 7 was to determine the performance of perforated shearwall construction as compared with the engineered construction of Phase 6. Recall that the intermediate HTT22 holdowns (located adjacent to the door openings) and the intermediate CS16 inter-story holdown straps (located adjacent to the second story windows) were removed from the east and west walls for the perforated shearwall construction. The structural responses of Phase 6 and Phase 7 are compared in terms of relative displacement time-histories, absolute acceleration time-histories, base shear force-displacement hysteresis loops, energy absorbed time-histories, peak anchor bolt forces, first story sill plate and holdown stud uplift, and second story sill plate slippage.

As a result of removing the intermediate holdowns, there was a significant increase in the roof relative displacement for the perforated shearwall construction. Figure 10.25 shows a comparison of the Level 4 roof relative displacement time-histories for the structure using engineered construction and structure using perforated shearwall construction. This increased lateral displacement was most likely due to an

![Graph comparing relative displacement time-histories for Phase 6 and Phase 7](image_url)

Figure 10.25: Comparison of Level 4 Roof Relative Displacement Time-Histories for Test Phase 6 and Test Phase 7
increase in sill plate uplift. In addition, there was a minor increase in the roof absolute acceleration with the perforated shearwall construction as shown in Figure 10.26.

![Figure 10.26: Comparison of Level 4 Roof Absolute Acceleration Time-Histories for Test Phase 6 and Test Phase 7](image)

The reduction in lateral stiffness of the test structure using perforated shearwall construction can be seen in a comparison of the Level 4 base shear force-displacement response as shown in Figure 10.27. In addition to an increased roof relative displacement, there was an increase in base shear for the test structure using perforated shearwall construction. There was a mild increase in the absorbed energy in the test structure for test Level 4 using perforated shearwall construction compared to engineered construction as shown in Figure 10.28. Although there was a large increase in lateral displacement, the majority of this increased displacement was most likely due to the removal of the holdowns adjacent to the door and window openings. As a result, there was not a significant increase in nail deformation in the shearwalls. Thus, there was not a large increase in absorbed energy for the test structure using perforated shearwall construction.
There was a redistribution of anchor bolt forces in the structure with the perforated shearwall construction compared with the engineered construction. Figure 10.29 shows a comparison of the peak anchor bolt forces for the structure using engineered construction and the structure using perforated shearwall construction. Since the perforated east and west shearwalls were designed using the entire 16-foot length with a reduction factor for the door opening, this design methodology will predict a lower holdown force at the shearwall ends as compared with engineered construction. However, as a result of removing the HTT22 holdowns adjacent to the...
door openings, the holdown forces at the ends of the east and west shearwall increased. This suggests that holdowns in perforated shearwall design should be design to a higher capacity than in segmented shearwall engineered design.

As noted previously in other comparisons, the sill plate and holdown stud uplift correlate well with the peak anchor bolt forces. Figure 10.30 shows a comparison of the Level 4 first story sill plate and holdown stud uplift for the structure using engineered construction and the structure using perforated shearwall construction. As would be expected, the sill plate and holdown stud uplift adjacent to the door openings was very high since the holdowns were removed and the nearest shear transfer anchor bolt was approximately 24 inches away from the edge of the door opening. The majority of this uplift adjacent to the door openings is a result of the holdown stud separating from the sill plate. The sill plate and holdown stud uplift were also significantly higher at the ends of the east and west walls as a result of the increase holdown forces. Again, the majority of the uplift at the wall ends was a result of holdown stud uplift. The sill plate and holdown stud uplift on the second story remained essentially the same. This suggests that the removal of the intermediate CS16 inter-story straps and the installation of additional sill plate nailing at the full-height shearwall segments as was shown in Figure 4.18 prevented additional sill plate uplift.

As a result of this increased nailing in the second story sill plates of the perforated shearwall design, the second story sill plate slippage on the east and west walls decreased. Figure 10.31 shows a comparison of the second story sill plate slippage for the structure using engineered construction and the structure using
Phase 6 – Engineered Construction

Phase 7 – Perforated Shearwall Construction

Figure 10.29: Comparison of Level 4 Peak Anchor Bolt Forces for Test Phase 6 and Test Phase 7
Phase 6 – Engineered Construction

Phase 7 – Perforated Shearwall Construction

Figure 10.30: Comparison of Level 4 First Story Peak Sill Plate and Holdown Stud Uplift for Test Phase 6 and Test Phase 7

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Phase 6 – Engineered Construction

Phase 7 – Perforated Shearwall Construction

Figure 10.31: Comparison of Level 4 Second Story Peak Sill Plate Slippage for Test Phase 6 and Test Phase 7
perforated shearwall construction. The sill plate slippage in the first story did not change with the perforated shearwall construction since the first story sill plate slippage is primarily a factor of the locations of the anchor bolts in the oversized holes.

**10.3.8 Effect of Anchor Bolt Locations with Perforated Shearwall Construction (Phases 7 & 7A)**

Since the sill plate and holdown stud uplift adjacent to the pedestrian door openings was significantly higher with the perforated shearwall construction, the shear transfer anchor bolts (4 total) that were 24 inches away from the door openings during Phase 7 were moved to 4 ½ inches away from the door openings for Phase 7A. The structural response of Phase 7 and Phase 7A are compared in terms of relative displacement time-histories, peak anchor bolt forces, and first story sill plate and holdown stud uplift.

There was essentially no increase in the relative displacement of the structure with the repositioning of these anchor bolts. Figure 10.32 shows a comparison of the

![Figure 10.32: Comparison of Level 4 Roof Relative Displacement Time-Histories for Test Phase 7 and Test Phase 7A](image-url)
Level 4 roof relative displacement for perforated shearwall construction with the shear transfer anchor bolts away from the door opening and the shear transfer anchor bolts at the door opening.

By moving the shear transfer anchor bolts closer to the door opening, the peak holdown force at the ends of the east and west walls increased. A comparison of the Level 4 peak anchor bolt forces for the structure with the shear transfer anchor bolts away from the door openings and the structure with the shear transfer anchor bolts at the door openings is shown in Figure 10.33. This increase in end-of-wall holdown forces is a result of making the shearwall elements on either side of the door openings more like segmented shearwalls. The shear transfer anchor bolt at the door opening prevented the sill from uplifting. Since the nailing of the wall sheathing ties the holdown stud to the sill plate, this arrangement acts as a soft “holdown” anchor. This soft “holdown” anchor works with the HTT22 holdown at the end of the wall to create a segmented shearwall. As a result, the holdown forces at the ends of the walls are higher.

Since the sill plate is restrained by the shear transfer anchor bolt at the door openings for Phase 7A, the sill plate uplift at these locations is significantly less. Figure 10.34 shows a comparison of the Level 4 sill plate and holdown stud uplift for the structure with the shear transfer anchor bolt away from the door openings and the structure with the shear transfer anchor bolts at the openings. Although the sill plate uplift decreased with the anchor bolt at the door opening, the holdown stud uplift remained nearly the same. In addition, the sill plate and holdown stud uplift at the ends of the east and west shearwalls remained the same.
Phase 7 – Anchor Bolt Away from Door Opening

Phase 7A – Anchor Bolt at Door Opening

Figure 10.33: Comparison of Level 4 Peak Anchor Bolt Forces for Test Phase 7 and Test Phase 7A
Phase 7 – Anchor Bolt Away from Door Opening

Phase 7A – Anchor Bolt at Door Opening

Figure 10.34: Comparison of Level 4 First Story Peak Sill Plate and Holdown Stud Uplift for Test Phase 7 and Test Phase 7A

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Since the sill plate was restrained at the door openings and the holdown stud was allowed to uplift for Phase 7A, excessive shear stresses developed in the wall sheathing at the corners of the door openings. Figure 10.35 shows an example of the typical cracking of the OSB wall sheathing that occurred at both corners of the two door openings during Phase 7A. This damage suggests that the shear transfer anchor bolts should be placed significantly far away from the door openings in perforated shearwall construction.

![Figure 10.35: Example of Cracking in Wall Sheathing at Corners of Door Openings During Test Phase 7A](image)

### 10.3.9 Performance of Conventional Construction (Phase 6 & 8)

In an effort to determine the performance of conventional construction (non-engineered) as compared with engineered construction, the test structure was modified during Phase 8 to represent conventional construction. All of the HTT22 holdowns and the CS16 inter-story holdown straps were removed from the east and west walls. In addition, the A35 shear transfer clips were removed from the east and west walls.
and replaced with additional second story sill plate nailing and floor diaphragm nailing through the blocking into the top plate of the first story walls. The structural response of Phase 6 and Phase 8 are compared in terms of relative displacement time-histories, absolute acceleration time-histories, peak anchor bolt forces, first story sill plate and holdown stud uplift, and second story sill plate slippage.

As a result of these modifications to represent conventional construction, there was a significant increase in the relative displacement during Phase 8. Figure 10.36 shows a comparison of the Level 4 roof relative displacement of the structure using engineered construction and conventional construction. Since the shear resistance of the shearwalls was not changed with the conventional construction, this increased displacement was most likely due to increase uplift and overturning. Although the relative displacement increased significantly with conventional construction, the Level 4 roof absolute acceleration remained the same as shown in Figure 10.37.

Figure 10.36: Comparison of Level 4 Roof Relative Displacement Time-Histories for Test Phase 6 and Test Phase 8
By removing the holdowns in the test structure using conventional construction, there was a significant decrease in lateral stiffness as shown by Level 4 base shear force-displacement hysteresis loops in Figure 10.38. The base shear remained essentially the same for the test structure using engineered construction and the test structure using conventional construction. Similar to the test structure using perforated shearwall construction, there was only a mild increase in the absorbed energy of the test structure using conventional construction as compared to the test structure using engineered construction. Figure 10.39 shows a comparison of the Level 4 energy absorbed time-histories for the test structure using engineered construction and the test structure using conventional construction. Since most of the increase in lateral displacement for the test structure using conventional construction was due to increased overturning, the nail deformation would not be increased significantly. This explains the only mild increase in absorbed energy for the test structure using conventional construction despite the large increase in displacement.
By removing the HTT22 holdowns and replacing them with shear transfer anchor bolts for the conventional construction, the anchor bolt forces increased in some locations and decreased in others. Figure 10.40 shows a comparison of the anchor bolt forces for the structure using engineered construction and conventional construction. Shear transfer anchor bolts at the end of each shearwall segment restrained the sill plates. Since the nailing of the wall sheathing tied the holdown stud to the sill plate, this arrangement acted as a soft “holdown” anchor. Depending on the relative stiffness of this soft “holdown”, the force in the shear transfer anchor bolt
could have been high or low. This explains the seemingly sporadic changes in the anchor bolt forces with the conventional construction.

Since the shear transfer anchor bolt restrained the sill plates at the end of the shearwall segments, the sill plate uplift at these locations was decreased in the convention construction as compared to the engineered construction. However, the holdown stud uplift at the shearwall segment ends increased in the conventional construction. Figure 10.41 shows the Level 4 first story peak sill plate and holdown stud uplift for the structure using engineered construction and the structure using conventional construction. This increased holdown stud uplift was probably a large contribution to the increased lateral displacement of the structure with conventional construction. As a result of the increased second story sill plate nailing for the structure with conventional construction, there was little change in the sill plate and holdown stud uplift on the second story.

The second story sill plate slippage decreased as a result of this increased sill plate nailing for the structure using conventional construction. Figure 10.42 shows a comparison of the second story peak sill plate slippage for the structure using engineered construction and the structure using conventional construction. However, the first story sill plate slippage remained the same since the first story slippage is dependent upon the anchor bolt locations in the oversized holes.

As was seen during Phase 7A when the shear transfer anchor bolts were placed very close to the door openings with the perforated shearwall design, significant shear stresses were developed in the wall sheathing at the corners of the door openings. Figure 10.43 shows a typical diagonal crack that developed in the OSB wall
Phase 6 – Engineered Construction

Phase 8 – Conventional Construction

Figure 10.40: Comparison of Level 4 Peak Anchor Bolt Forces for Test Phase 6 and Test Phase 8
Phase 6 – Engineered Construction

Phase 8 – Conventional Construction

1st Story Uplift Configuration

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.

Figure 10.41: Comparison of Level 4 First Story Peak Sill Plate and Holdown Stud Uplift for Test Phase 6 and Test Phase 8
Phase 6 – Engineered Construction

Phase 8 – Conventional Construction

☐ Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

☐ Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to floor sheathing on 2nd story. All values are in inches.

Figure 10.42: Comparison of Level 4 Second Story Peak Sill Plate Slippage for Test Phase 6 and Test Phase 8
sheathing at the corners of the door openings with the conventional construction. As with Phase 7A, these high shear stresses that developed at the door opening corners were most likely due to the shear transfer anchor bolts being located too close to the shearwall segment ends.

Figure 10.43: Example of Cracking in Wall Sheathing at Corners of Door Openings During Test Phase 8

10.3.10 Effect of Wall Finish Materials (Phase 9 and Phase 10)

For the final phase of testing (Phase 10), interior gypsum wallboard and exterior stucco was installed on the test structure to evaluate the effect of these wall finish materials on the seismic performance of the structure. The ½ inch interior gypsum wallboard was installed on all interior wall and ceiling surfaces and was mudded, taped, and painted. The exterior stucco was applied as a three-coat system producing a total thickness of 7/8 inch. A pedestrian door was installed in the first story west wall and aluminum windows were installed in all walls as specified in the plans. No door was installed in the garage door opening of the first story east wall.
Recall from Chapter 4 that most of the supplemental weights used in previous testing phases were removed for test Phase 10 because of the additional weight of the stucco and gypsum wallboard. The structural responses of Phase 9 and Phase 10 are compared in terms of relative displacement time-histories, absolute acceleration time-histories, base shear force-displacement hysteresis loops, energy absorbed time-histories, peak anchor bolt forces, first story sill plate and holdown stud uplift, and second story sill plate slippage.

The non-structural finish materials had a significant effect of the seismic performance of the test structure. The Level 5 roof relative displacement decreased by more than a factor of three with the wall finish materials during Phase 10 as shown in Figure 10.44. This decrease in displacement response can be attributed to a large increase in lateral stiffness of the structure with the wall finish materials. The frequency evaluation test results showed that the fundamental frequency of the structure with no wall finish materials was 2.93 Hz before Level 5, and increased to 5.47 Hz before Level 5 with the wall finish materials. Using a SDOF model assumption, this increase in fundamental frequency corresponds to an increase of 3.5 in lateral stiffness. Note also that the roof residual displacement of 0.29 inch in the structure without wall finish materials was eliminated in the structure incorporating the wall finish materials.

As a result of the stiffening of the structure with wall finish materials, the acceleration response increased moderately. Figure 10.45 shows a comparison of the Level 5 roof absolute acceleration time histories for the structure without wall finish materials and the structure without wall finish materials.
A comparison of base shear force-displacement hysteresis loops for the structure with and without wall finish materials shows very different seismic responses as shown in Figure 10.46 for test Level 4 and Figure 10.48 for test Level 5. As noted previously in the discussion on capacity spectra, the behavior of the test structure with wall finish materials was nearly elastic. In addition, these base shear force-displacement hysteresis loops show a significant increase in lateral stiffness for the test structure with wall finish materials. Since the test structure remained nearly elastic
with wall finish materials, the absorbed energy decreased significantly as shown in Figure 10.47 for test Level 4 and Figure 10.49 for test Level 5.

Figure 10.46: Comparison of Level 4 Base Shear Force-Displacement Hysteresis Loops for Test Phase 9 and Test Phase 10

Figure 10.47: Comparison of Level 4 Energy Absorbed Time-Histories for Test Phase 9 and Test Phase 10

Figure 10.48: Comparison of Level 5 Base Shear Force-Displacement Hysteresis Loops for Test Phase 9 and Test Phase 10
There was a significant redistribution of anchor bolt forces in the structure incorporating wall finish materials. Figure 10.50 shows a comparison of the peak anchor bolt forces in the test structure without wall finish materials and the structure with wall finish materials. The very high holdown forces in the east and west shearwalls during Phase 9 were significantly reduced by the introduction of the wall finish materials during Phase 10. In addition, there was an increase in anchor bolt forces in the perpendicular walls (particularly in the south wall) with the wall finish materials. This redistribution of anchor bolt forces into the perpendicular walls resulted from the structure behaving as a shell due to the presence of the wall finish materials.

The wall finish materials also had a significant effect on the sill plate and holdown stud uplift. Figure 10.51 shows a comparison of the Level 5 first story peak sill plate and holdown stud uplift for the test structure without wall finish materials and the structure with wall finish materials. Both the sill plate uplift and holdown stud
uplift are reduced in the structure with wall finish materials. The reduction in the holdown stud uplift can be attributed to the interior gypsum wallboard and the exterior stucco. The reduction in the sill plate uplift was probably a result of the structure behaving as a rigid shell with the wall finish materials. There was little reduction in the second story sill plate and holdown stud uplift since it was already very low without wall finish materials.

There was a large reduction in sill plate slippage at the second story of the structure with the wall finish materials. A comparison of the Level 5 sill plate slippage for the structure without wall finish materials and the structure with wall finish materials is shown in Figure 10.52. The large reduction in sill plate slippage was probably a result of the exterior stucco resisting this slippage at the floor diaphragm level.
Phase 9 – Without Wall Finish Materials

Phase 10 – With Wall Finish Materials

Figure 10.50: Comparison of Level 5 Peak Anchor Bolt Forces for Test Phase 9 and Test Phase 10
Phase 9 – Without Wall Finish Materials

Phase 10 – With Wall Finish Materials

Figure 10.51: Comparison of Level 5 First Story Peak Sill Plate and Holdown Stud Uplift for Test Phase 9 and Test Phase 10

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Phase 9 – Without Wall Finish Materials

Phase 10 – With Wall Finish Materials

Note: Sill plate slippage measured relative to floor sheathing on 2nd story. All values are in inches.

Figure 10.52: Comparison of Level 5 Second Story Peak Sill Plate Slippage for Test Phase 9 and Test Phase 10
10.4 Summary of Seismic Test Results

The seismic test results have provided an opportunity to evaluate the effects of various structural configurations on the seismic performance of the structure. The major findings from the seismic tests that could influence future design and construction practices for woodframe construction are summarized below:

- Seismic tests under repeated moderate ground motions showed that there could be a significant degradation in the seismic performance of the structure if structural damage was not repaired following the first moderate earthquake.
- Symmetrical window and door openings caused a significant increase in the relative lateral displacement compared to a fully sheathed structure due to a reduction in lateral stiffness and an increase in overturning.
- Non-symmetrical openings caused a significant torsional response in the structure because the first story shearwalls each had a different stiffness and the floor diaphragm behaved rigidly. The torsional deformation of the structure can also be seen in the anchor bolt forces, sill plate and holdown stud uplift, and sill plate slippage in the north and south walls.
- The removal of the waste wall sheathing above and below window openings caused a significant reduction in the lateral stiffness of the structure. Since the shearwalls behaved as short shearwall segments without the waste wall sheathing, there was increased overturning. The combination of increased overturning and reduction in lateral stiffness caused a significant increase in the lateral displacement of the structure.
• There was a significant increase in lateral displacement in the structure using perforated shearwall design primarily due to increased sill plate and holdown stud uplift.

• The perforated shearwall design methodology used to predict the holdown force requirements at the wall ends appears to be unconservative. In fact, the holdown forces at the wall ends were higher than with segmented shearwall construction.

• There was an increased lateral displacement for the structure using conventional construction as a result of holdown stud uplift at the shearwall segment ends.

• With perforated shearwall construction and conventional construction, the shear transfer anchor bolts should be placed sufficiently far enough away from the door openings to prevent shear stress concentrations at the door opening corners and also to prevent an increase in holdown forces at the perforated shearwall ends.

• The displacement response of the structure was significantly reduced with the wall finish materials due to the large increase in stiffness of the structure.

• The structure behaved nearly as an elastic shell when the wall finish materials were incorporated as shown by the redistribution of anchor bolt forces and sill plate and holdown stud uplift.

• Changes in the sill plate and holdown stud uplift closely followed changes in peak anchor bolt forces for many of the configurations.
• The second story sill plate slippage was dependent on the nailing of the bottom wall plate to the rim joists and the presence of wall finish materials.

• The HTT22 and CS16 holdown devices used caused excessive splitting of the wood (due to the close nail spacing).
Chapter 11 Numerical Modeling of Test Structure

In an effort to numerically predict the results of the shake table tests, the Phase 9 test structure was modeled using the non-linear dynamic time-history analysis program RUAUMOKO (Carr, 1998). The Phase 9 test structure incorporating a garage door opening in the first story east wall and a pedestrian door opening in the first story west wall was considered for modeling purposes as was shown in Figure 4.25. The two-story test structure was degenerated into a 2-dimensional pancake model. The shearwalls of the test structure were represented by equivalent single-degree-of-freedom (SDOF) nonlinear shear elements (springs) using hysteretic parameters developed from the cyclic analysis of wood shearwalls program CASHEW (Folz and
Filiatrault, 2000) that used sheathing-to-framing connection test data. In this chapter, the numerical pancake model used in RUAUMOKO is discussed. In addition, a brief description of the CASHEW program is presented along with a description of how the hysteretic parameters for the shearwalls were obtained. The response predictions obtained at different locations in the test structure from static pushover and nonlinear dynamic time history analyses are then compared with the associated experimental responses.

11.1 Description of Pancake Model

The test structure was modeled as a planar pancake system with the floor diaphragm and roof diaphragm superimposed on top of each other. The foundation of the structural model was connected to the floor diaphragm with four zero-length nonlinear shear springs representing the four first story shearwalls, as shown in Figure 11.1. Similarly, the floor diaphragm was connected to the roof diaphragm with four additional zero-length nonlinear shear springs representing the four second story shearwalls. The roof diaphragm was assumed rigid and modeled by four plane stress quadrilateral finite elements with very high in-plane stiffness. The floor diaphragm was modeled by 34 plane stress quadrilateral finite elements. The in-plane stiffness of the floor diaphragm was calibrated using the quasi-static test results that were discussed previously in Chapter 7. Using a constant elastic modulus for the quadrilateral elements, a concentrated lateral load was applied to the center of the floor diaphragm model, equal to the maximum load (7.97 kips) during Quasi-Static Test 9.Q. The thickness of the quadrilateral elements of the floor diaphragm was adjusted such that the relative deformation of the floor diaphragm in the numerical
model equaled the relative deformation of the floor diaphragm in the experimental quasi-static test (0.049 inches). Table 11.1 shows the member properties that were used for all elements in the numerical model.

Frame elements were used along the four edges of the floor diaphragm to connect the corners of the quadrilateral elements to the shear elements. The bending stiffness of the frame elements was assumed very small to allow free deformations of the diaphragm. The axial stiffness of the frame elements was assumed very high

Figure 11.1: Numerical Pancake Model
Table 11.1: Member Properties for Numerical Model

<table>
<thead>
<tr>
<th>Component</th>
<th>Section</th>
<th>Elastic Modulus (ksi)</th>
<th>Moment of Inertia (in^4)</th>
<th>Area (in^2)</th>
<th>Thickness (in)</th>
<th>Poisson's Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof Diaphragm</td>
<td>Plane Stress Quadrilateral Elements</td>
<td>1,700</td>
<td>--</td>
<td>--</td>
<td>100</td>
<td>0.3</td>
</tr>
<tr>
<td>Floor Diaphragm</td>
<td>Plane Stress Quadrilateral Elements</td>
<td>1,700</td>
<td>--</td>
<td>--</td>
<td>0.0915</td>
<td>0.3</td>
</tr>
<tr>
<td>Floor Diaphragm Constraining Beams (All Edges)</td>
<td>Frame Elements</td>
<td>1,700</td>
<td>0.001</td>
<td>1 x 10^6</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Quasi-Static Loading Channel (C6 x 8.2)</td>
<td>Frame Elements</td>
<td>29,000</td>
<td>13.1</td>
<td>2.40</td>
<td>--</td>
<td>--</td>
</tr>
</tbody>
</table>

in order to distribute the in-plane forces of the shear elements along the edges of the floor diaphragm. Since a C6 x 8.2 channel section was attached to the floor diaphragm for the quasi-static tests on the experimental structure, frame elements with properties of the channel section were used along the center of the floor diaphragm to constrain the deformation of the nodes of the quadrilateral elements at the center of the diaphragm.

Each of the eight shearwalls in the test structure was modeled by a single zero-length nonlinear spring using the Wayne Stewart degrading stiffness hysteresis (Stewart, 1987). Figure 11.2 shows this hysteresis rule and the required input parameters for RUAUMOKO. The parameters used to define the shearwall hysteretic parameters were developed using the CASHEW program and will be discussed in detail in the following section.
The weight of the test structure for use in the model was computed using the weights of the individual framing members, the weight of floor, roof, and wall sheathing panels, the weight of the clay roof tiles, and the weight of the supplemental weights. The total weight of the structure used was 24.58 kips with 13.83 kips at the floor diaphragm and 10.75 kips at the roof diaphragm. The weight of the upper half of the second story walls was distributed along the edges of the roof diaphragm. The weight of the lower half of the second story walls and the upper half of the first story walls was distributed along the edges of the floor diaphragm. These wall weights were distributed to the edge nodes based on the spacing of these nodes. The weight of the supplemental weights on the upper half of the second story walls was distributed to the
roof diaphragm edge nodes (see Appendix B for the exact locations of the supplemental weights). The entire roof weight (trusses, sheathing, clay tiles, supplemental roof weights) was lumped at the center node of the diaphragm. A mass moment of inertia at this center node was used to account for the distribution of this weight. The total floor weight (framing, sheathing, second story partitions, and floor supplemental weights) was computed and then distributed evenly to the nodes of the floor diaphragm based on the tributary area of these nodes.

For the RUAUMOKO model, the percentage of critical damping was set at 7.6% for the first and second modes using Rayleigh damping. As discussed in Chapter 9, this damping value corresponds to the mean damping value determined from the damping evaluation tests of the test structure.

The recorded acceleration time histories of the shake table for test Phase 9 (Channel B1) were used as the input ground accelerations for the RUAUMOKO model. Since the experimental structure was tested at five levels (including a repeat of Level 3), the recorded acceleration time histories were assembled into a train of ground motions for the dynamic time-history analysis of the structure. A minimum time gap of five seconds was used between each of these ground motions to allow free vibration in the structure. By using the train of ground motions, the accumulated hysteretic damage could be taken into account for each level of seismic shaking. Figure 11.3 shows the train of acceleration records used in the numerical model with the positive and negative peak input accelerations for each seismic level.
11.2 Shearwall Hysteretic Parameters

The CASHEW program (Folz and Filiatrault, 2000) was used in conjunction with the connection test data shown in Appendix D to determine the hysteretic parameters for the nonlinear shear springs used to model the shearwall elements in RUAUMOKO. CASHEW can predict the cyclic load-displacement response of sheathed shearwalls with or without openings. In CASHEW, each shearwall is modeled using three structural components: rigid framing members, linear elastic sheathing panels, and nonlinear sheathing-to-framing connectors. Using the configuration of each shearwall (dimensions of shearwall, number and size of openings, and nail spacing) along with cyclic connection test data (discussed previously in Chapter 6), the CASHEW program was able to predict the hysteretic response under the CUREe-Caltech cyclic loading protocol presented previously in
Figure 6.2. The CASHEW program was able to calibrate an equivalent single-degree-of-freedom (SDOF) system for the nonlinear hysteretic response of the shearwalls. Figure 11.1 shows the equivalent SDOF model prediction of the hysteretic response of the first story east shearwalls developed from CASHEW.

A similar equivalent SDOF model prediction of the hysteretic response of each shearwall in the test structure was developed. From these predictions, the parameters that define the hysteretic response were determined for use in the Wayne Stewart hysteresis rule (see Figure 11.2). This hysteretic model was used for each nonlinear shear spring of the test structure in RUAUMOKO. Figure 11.5 shows the load-displacement hysteresis loops for the shearwalls used in the numerical model that were
Figure 11.5: Load-Displacement Hysteresis Loops of Shearwalls for Numerical Model
determined from the CASHEW program. Table 11.2 shows the hysteretic parameters determined from the equivalent SDOF CASHEW model prediction that were used in the Wayne Stewart Hysteresis for each of the shearwalls in the RUAUMOKO model.

Table 11.2: Wayne Stewart Hysteresis Parameters Developed from CASHEW

<table>
<thead>
<tr>
<th>Parameter</th>
<th>1st Story Shearwalls</th>
<th>2nd Story Shearwalls</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>North &amp; South</td>
<td>East</td>
</tr>
<tr>
<td>Initial Stiffness, $k_0$ (kips/in)</td>
<td>26.3</td>
<td>8.42</td>
</tr>
<tr>
<td>Post-Yield Stiffness Factor, $R_f$</td>
<td>0.125</td>
<td>0.115</td>
</tr>
<tr>
<td>Post-Ultimate Stiffness Factor, $P_{TRI}$</td>
<td>-0.086</td>
<td>-0.068</td>
</tr>
<tr>
<td>Unloading Stiffness Factor, $P_{UNL}$</td>
<td>1.10</td>
<td>1.06</td>
</tr>
<tr>
<td>Softening Factor, $\beta$</td>
<td>1.09</td>
<td>1.08</td>
</tr>
<tr>
<td>Reloading Factor, $\alpha$</td>
<td>0.807</td>
<td>0.767</td>
</tr>
<tr>
<td>Yield Force, $F_y$ (kips)</td>
<td>10.5</td>
<td>4.40</td>
</tr>
<tr>
<td>Ultimate Force, $F_u$ (kips)</td>
<td>16.0</td>
<td>7.00</td>
</tr>
<tr>
<td>Intercept Force, $F_i$ (kips)</td>
<td>2.44</td>
<td>0.99</td>
</tr>
</tbody>
</table>

11.3 Pushover Analysis

A static pushover analysis was performed using RUAUMOKO to determine whether the experimental test structure was near its ultimate capacity. Figure 11.6 shows capacity spectra for the experimental test structure and the dynamic numerical model along with a static pushover analysis. The base shear for the experimental and numerical capacity spectra was computed as the sum of the inertia forces at the floor and roof levels. The capacity spectrum for the experimental test structure is identical to that presented in Chapter 10 for test Phase 9. Note that this base shear is the total force induced in the foundation of the test structure including the nonlinear restoring force and the viscous damping force. However, the capacity spectrum developed from
the static pushover analysis represents only the nonlinear restoring force since the velocity is zero (resulting in no damping force).

The experimental and numerical capacity spectra are in agreement for test Levels 1 to 3, but the numerical model under predicts the capacity and displacement for test Levels 4 and 5. However, both the experimental and numerical results for test Levels 4 and 5 show that the structure has not reached its ultimate capacity. The capacity spectra developed from the static pushover analysis is significantly lower than both the experimental and numerical capacity spectra for test Levels 4 and 5 since the viscous damping force is not included. It can be seen from the static pushover analysis that the ultimate capacity of the structure is reached at a peak roof relative

![Graph](image-url)

Figure 11.6: Capacity Spectra for Experimental Test Structure and Numerical Model
displacement of approximately 5¼ inches.

11.4 **Comparison of Fundamental Frequencies and Mode Shapes**

The fundamental frequency and the fundamental structural mode shape were determined for the numerical model using the modal analysis option in RUAUMOKO. Figure 11.7 shows a comparison of the fundamental frequencies and mode shapes for the experimental test structure and the numerical model. The fundamental frequency of the experimental test structure (3.96 Hz) was significantly higher than the fundamental frequency predicted by the numerical model (2.69 Hz). This experimental natural frequency corresponds to the initial stiffness of the test structure before test Phase 9 (Frequency Test 9.F.0 in Appendix F). Since the numerical model used an initial elastic stiffness for the shearwall elements, this initial stiffness predicted by the numerical model was lower than the experimental initial stiffness resulting in the lower fundamental frequency. Although there is a discrepancy in the numerical and experimental fundamental frequencies, the numerical model predicts the fundamental mode shape reasonable well as shown in Figure 11.7. The modal values at the floor level of the numerical model for both the east and west walls were slightly higher than the experimental results. This increased modal value suggests a lower lateral stiffness in the first story shearwalls in the numerical model. Again, the lower lateral stiffness was probably the result of using an initial elastic stiffness in the numerical model.
**Experimental**

*Fundamental Frequency = 3.96 Hz*

![East Elevation](image1)

![West Elevation](image2)

**Numerical**

*Fundamental Frequency = 2.69 Hz*

![East Elevation](image3)

![West Elevation](image4)

Figure 11.7: Comparison of Experimental and Numerical Fundamental Frequencies and Mode Shapes for Test Phase 9
11.5 Comparison of Numerical Results with Experimental Results for Test Phase 9

Relative displacement and acceleration time-histories from the numerical model along with the corresponding experimental time-histories for the first and second story east and west shearwalls are presented in Appendix O. These results show a good correlation between the numerical model and the experimental results for the higher level amplitude ground motions. Comparisons of the relative displacement, acceleration time-histories, and global hysteresis loops for Phase 9 are presented in the following two sections.

11.5.1 Displacement Time-Histories

For the lower amplitude levels of Phases 9, the numerical model over predicted the relative displacement by a factor between two to three. Figure 11.8 shows a comparison of the experimental and numerical relative displacement time-histories for the first story east wall for Level 1. This over prediction of the relative displacement may have been the result of using an initial linear stiffness for the hysteretic properties of the shearwalls at low amplitudes.

![Figure 11.8: Comparison of Experimental and Numerical Relative Displacement Time-Histories for First Story East Wall for Phase 9, Level 1](image-url)
The correlation between the numerical and experimental relative displacement
time-histories was good for the higher amplitude levels. Figure 11.9 and Figure 11.10
show the experimental and numerical relative displacement time-histories for the first
story east wall for Level 4 and Level 5, respectively. The peak relative displacements
for the experimental and numerical model are fairly close. In addition, experimental
and numerical relative displacement time-histories are in phase with one another. It
was also found that the numerical model over estimates the relative displacement on
the first story, but under estimate the relative displacement on the second story. This
may have been the result of neglecting overturning effects in the numerical model.

![Figure 11.9: Comparison of Experimental and Numerical Relative Displacement Time-Histories for First Story East Wall for Phase 9, Level 4](image)

![Figure 11.10: Comparison of Experimental and Numerical Relative Displacement Time-Histories for First Story East Wall for Phase 9, Level 5](image)
11.5.2 Acceleration Time-Histories

Unlike the relative displacement time-histories at low amplitude levels, the numerical model accurately predicted the peak absolute acceleration for low amplitude levels. Figure 11.11 shows a comparison of the experimental and numerical absolute acceleration time-histories for the first story east wall for Level 1. For these time-histories, the peak values correspond well, but the time-histories are not in phase with one another. Again, this was most likely a result of the model using an initial linear stiffness for the shearwall hysteresis properties.

The absolute acceleration time-histories for the numerical model is in reasonable agreement with the actual recorded absolute acceleration for the higher amplitude levels. Similar to the relative displacement time-histories, the numerical model slightly over predicts the absolute acceleration on the first story, but the absolute acceleration time histories of the numerical model and the experimental data are in phase with one another. Figure 11.12 and Figure 11.13 show a comparison of the experimental and numerical absolute acceleration time-histories for the first story.
east wall for Level 4 and Level 5, respectively. This comparison shows that the numerical model over predicted the peak absolute acceleration, but the acceleration time-histories from the numerical model and experimental data are in phase with one another. Although the numerical model over predicted the absolute acceleration at the first story, the numerical model under predicted the absolute acceleration at the second story. As was stated previously, this may have been the result of neglecting overturning effects in the model.
11.5.3 Global Hysteretic Behavior

At the low levels of seismic testing, the lower initial stiffness discussed previously can be seen in the global hysteretic behavior of the numerical model. Figure 11.14 shows a comparison of the experimental and numerical global hysteretic behavior of the structure for Level 1 of test Phase 9. Although the numerical model agrees with the experimental base shear, the numerical model overestimates the roof displacement as a result of the lower initial stiffness.

![Figure 11.14: Comparison of Global Hysteretic Behavior for First Story East Wall for Phase 9, Level 1](image)

The numerical model captures the general shape of the global hysteresis loops for Levels 4 and 5 of test Phase 9 as shown in Figure 11.15 and Figure 11.16, respectively. However, the numerical model underestimates the base shear for Levels 4 and 5. The numerical model captures the first major positive pulse for the Canoga Park ground motion of test Level 4, but fails to capture the following major negative pulse. The major negative pulse in the experimental structure may have been caused by overturning or any type of slip that the numerical model did not consider. Similarly, the numerical model captures the first major negative pulse for the Rinaldi
ground motion for test Level 5, but fails to capture the following major positive pulse. Again, this may have been caused by the numerical modeling not being able to consider overturning and slip.

**Figure 11.15: Comparison of Global Hysteretic Behavior for First Story East Wall for Phase 9, Level 4**

**Figure 11.16: Comparison of Global Hysteretic Behavior for First Story East Wall for Phase 9, Level 5**

### 11.5.4 Discussion on Numerical Modeling

From these numerical analyses, it was shown that the simple numerical model considered produced relatively good results in predicting the seismic response of the test structure. It was shown that the numerical model predicts the seismic response well for large amplitude events. However, the numerical model tended to over predict the seismic response for low amplitude events due to the initial linear stiffness that
was used for the shearwall hysteresis properties. These results indicated that it might not be necessary to develop very extensive finite element models to accurately predict the behavior of a woodframe structure under a seismic event. One should note that the predictions of this simple numerical model were evaluated only with the relatively simple test structure of Task 1.1.1. Similar numerical models should be developed and assessed against more complicated structures.
Chapter 12 Summary and Conclusions

12.1 Summary

Under Task 1.1.1 of the CURIEe-Caltech Woodframe Project, a two-story single-family woodframe house was tested using the UC San Diego uniaxial earthquake simulation system to determine the dynamic characteristics and seismic performance of the structure. The two-story test structure was 16 feet wide by 20 feet long and oriented such that shaking occurred along the short dimension of the structure. The test structure was constructed using two-by-four construction with the exterior walls acting as shearwalls and sheathed with OSB structural panels. The structure was tested during 10 different phases of construction including a structure
with fully sheathed shearwalls, symmetrical window and door openings, unsymmetrical window and door openings, perforated shearwall construction, conventional construction, and with and without non-structural wall finish materials. Four types of shake table tests were performed on the test structure: quasi-static in-plane floor diaphragm tests, frequency evaluation tests, damping evaluation tests, and seismic tests. For each phase of testing, up to five levels of seismic tests were performed (increasing in amplitude with each test level) using an ordinary ground motion (1994 Northridge Earthquake at Canoga Park) and a near fault ground motion (1994 Northridge Earthquake at Rinaldi). A numerical model, based on non-linear time-history dynamic analyses, was also developed to predict the seismic response of the structure and then compared to the experimental response of the test structure.

12.2 Conclusions

From the quasi-static floor diaphragm tests, frequency evaluation tests, damping evaluation tests, and seismic tests performed on the test structure, conclusions have been drawn regarding the dynamic characteristics of the structure and the effects of different structural configurations of the seismic performance of the structure. The key results obtained are summarized below:

- The installation of floor diaphragm blocking at unsupported plywood panel edges and subfloor adhesive caused a significant increase of the in-plane shear stiffness of the floor diaphragm.
- The second story walls caused a significant increase of the in-plane flexural stiffness of the floor diaphragm.
The presence of the interior and exterior wall finish materials caused a large increase of the overall in-plane stiffness of the floor diaphragm.

Nearly all tests (quasi-static and seismic) showed the diaphragm behaving rigidly according to UBC Section 1630.6.

The initial fundamental frequency of the test structure ranged from 3.96 Hz with unsymmetrical openings (Phase 9) to 6.49 Hz with unsymmetrical openings and non-structural wall finish materials (Phase 10).

For all testing phases, there was a significant reduction in fundamental frequency of the structure between the Level 3 (Canoga Park, 0.36g) and Level 4 (Canoga Park, 0.50 g) seismic tests.

There were significant variations in the measured equivalent viscous damping. The mean damping for all tests was 7.6% of critical.

Seismic tests with repeated moderate ground motions showed there was a significant degradation in the seismic performance of the structure if structural damage was not repaired following the first moderate earthquake.

Symmetrical window and door openings caused a significant increase in the relative lateral displacement compared to a fully sheathed structure due to a reduction in lateral stiffness and an increase in overturning.

Non-symmetrical openings caused a significant torsional response of the structure due to the different lateral stiffness of the first story shearwalls.

The removal of the waste wall sheathing above and below window openings caused a significant reduction in the lateral stiffness of the structure.
• There was a significant increase in lateral displacement of the structure using perforated shearwall design as compared with engineered construction.

• The perforated shearwall design methodology used to predict the holdown force requirements at the wall ends appears to be unconservative. In fact, the holdown forces at the wall ends were higher than with segmented shearwall construction.

• There was an increase in lateral displacement of the structure using conventional construction as compared with engineered construction.

• With perforated shearwall construction and conventional construction, the shear transfer anchor bolts should be placed sufficiently far enough away from the door openings to prevent shear stress concentrations at the door opening corners and also to prevent an increase in holdown forces at the perforated shearwall ends.

• The displacement response of the structure with wall finish materials was significantly reduced and the acceleration response was moderately increased as compared to the structure without wall finish materials. The structure incorporating wall finish materials behaved as a shell with increased lateral stiffness.

• The HTT22 and CS16 holdown devices used caused excessive splitting of the wood (due to the close nail spacing).

• The proposed numerical model was able to predict with reasonable accuracy the global seismic response of the test structure.
Chapter 13 Appendices
Appendix A  Architectural and Structural Drawings of Test Structure
Appendix B  Description of Experimental Tests
## Appendix B

Table 13.1: Tests Phases 1 – 4

<table>
<thead>
<tr>
<th>Test Designation</th>
<th>Phase</th>
<th>Test Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.Q</td>
<td>1</td>
<td>Quasi-Static</td>
</tr>
<tr>
<td>2.Q</td>
<td>2</td>
<td>Quasi-Static</td>
</tr>
<tr>
<td>3A.Q</td>
<td>3A</td>
<td>Quasi-Static</td>
</tr>
<tr>
<td>3B.Q</td>
<td>3B</td>
<td>Quasi-Static</td>
</tr>
<tr>
<td>4A.Q</td>
<td>4A</td>
<td>Quasi-Static</td>
</tr>
<tr>
<td>4B.Q</td>
<td>4B</td>
<td>Quasi-Static</td>
</tr>
<tr>
<td>4C.Q</td>
<td>4C</td>
<td>Quasi-Static</td>
</tr>
<tr>
<td>4D.Q</td>
<td>4D</td>
<td>Quasi-Static</td>
</tr>
</tbody>
</table>

Table 13.2: Test Phase 5

<table>
<thead>
<tr>
<th>Test Designation</th>
<th>Test Type</th>
<th>Test Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.Q</td>
<td>Quasi-Static</td>
<td>-</td>
</tr>
<tr>
<td>5.F.0</td>
<td>Frequency</td>
<td>0</td>
</tr>
<tr>
<td>5.F.1</td>
<td>Frequency</td>
<td>1</td>
</tr>
<tr>
<td>5.F.2</td>
<td>Frequency</td>
<td>2</td>
</tr>
<tr>
<td>5.F.3</td>
<td>Frequency</td>
<td>3</td>
</tr>
<tr>
<td>5.F.4</td>
<td>Frequency</td>
<td>4</td>
</tr>
<tr>
<td>5.F.4R</td>
<td>Frequency</td>
<td>4R</td>
</tr>
<tr>
<td>5.D.0</td>
<td>Damping</td>
<td>0</td>
</tr>
<tr>
<td>5.D.1</td>
<td>Damping</td>
<td>1</td>
</tr>
<tr>
<td>5.D.2</td>
<td>Damping</td>
<td>2</td>
</tr>
<tr>
<td>5.D.3</td>
<td>Damping</td>
<td>3</td>
</tr>
<tr>
<td>5.D.4</td>
<td>Damping</td>
<td>4</td>
</tr>
<tr>
<td>5.D.4R</td>
<td>Damping</td>
<td>4R</td>
</tr>
<tr>
<td>5.S.1</td>
<td>Seismic</td>
<td>1</td>
</tr>
<tr>
<td>5.S.2</td>
<td>Seismic</td>
<td>2</td>
</tr>
<tr>
<td>5.S.3</td>
<td>Seismic</td>
<td>3</td>
</tr>
<tr>
<td>5.S.4</td>
<td>Seismic</td>
<td>4</td>
</tr>
<tr>
<td>5.S.4R</td>
<td>Seismic</td>
<td>4R</td>
</tr>
</tbody>
</table>

Note: An "R" followed by the test level denotes a repeat test of a particular test level.
Table 13.3: Test Phase 6

<table>
<thead>
<tr>
<th>Test Designation</th>
<th>Test Type</th>
<th>Test Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.Q</td>
<td>Quasi-Static</td>
<td>-</td>
</tr>
<tr>
<td>6.F.0</td>
<td>Frequency</td>
<td>0</td>
</tr>
<tr>
<td>6.F.1</td>
<td>Frequency</td>
<td>1</td>
</tr>
<tr>
<td>6.F.2</td>
<td>Frequency</td>
<td>2</td>
</tr>
<tr>
<td>6.F.3</td>
<td>Frequency</td>
<td>3</td>
</tr>
<tr>
<td>6.F.4</td>
<td>Frequency</td>
<td>4</td>
</tr>
<tr>
<td>6.F.4R</td>
<td>Frequency</td>
<td>4R</td>
</tr>
<tr>
<td>6.D.0</td>
<td>Damping</td>
<td>0</td>
</tr>
<tr>
<td>6.D.1</td>
<td>Damping</td>
<td>1</td>
</tr>
<tr>
<td>6.D.2</td>
<td>Damping</td>
<td>2</td>
</tr>
<tr>
<td>6.D.3</td>
<td>Damping</td>
<td>3</td>
</tr>
<tr>
<td>6.D.4</td>
<td>Damping</td>
<td>4</td>
</tr>
<tr>
<td>6.D.4R</td>
<td>Damping</td>
<td>4R</td>
</tr>
<tr>
<td>6.S.1</td>
<td>Seismic</td>
<td>1</td>
</tr>
<tr>
<td>6.S.2</td>
<td>Seismic</td>
<td>2</td>
</tr>
<tr>
<td>6.S.3</td>
<td>Seismic</td>
<td>3</td>
</tr>
<tr>
<td>6.S.4</td>
<td>Seismic</td>
<td>4</td>
</tr>
<tr>
<td>6.S.4R</td>
<td>Seismic</td>
<td>4R</td>
</tr>
</tbody>
</table>

Note: An “R” followed by the test level denotes a repeat test of a particular test level

Table 13.4: Test Phase 6A

<table>
<thead>
<tr>
<th>Test Designation</th>
<th>Test Type</th>
<th>Test Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.A.F.2</td>
<td>Frequency</td>
<td>2</td>
</tr>
<tr>
<td>6.A.F.3</td>
<td>Frequency</td>
<td>3</td>
</tr>
<tr>
<td>6.A.D.2</td>
<td>Damping</td>
<td>2</td>
</tr>
<tr>
<td>6.A.D.3</td>
<td>Damping</td>
<td>3</td>
</tr>
<tr>
<td>6.A.S.3</td>
<td>Seismic</td>
<td>3</td>
</tr>
</tbody>
</table>
### Appendix B

#### Table 13.5: Test Phase 7

<table>
<thead>
<tr>
<th>Test Designation</th>
<th>Test Type</th>
<th>Test Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.Q</td>
<td>Quasi-Static</td>
<td>-</td>
</tr>
<tr>
<td>7.F.0</td>
<td>Frequency</td>
<td>0</td>
</tr>
<tr>
<td>7.F.1</td>
<td>Frequency</td>
<td>1</td>
</tr>
<tr>
<td>7.F.2</td>
<td>Frequency</td>
<td>2</td>
</tr>
<tr>
<td>7.F.3</td>
<td>Frequency</td>
<td>3</td>
</tr>
<tr>
<td>7.F.3R</td>
<td>Frequency</td>
<td>3R</td>
</tr>
<tr>
<td>7.F.4</td>
<td>Frequency</td>
<td>4</td>
</tr>
<tr>
<td>7.D.0</td>
<td>Damping</td>
<td>0</td>
</tr>
<tr>
<td>7.D.1</td>
<td>Damping</td>
<td>1</td>
</tr>
<tr>
<td>7.D.2</td>
<td>Damping</td>
<td>2</td>
</tr>
<tr>
<td>7.D.3</td>
<td>Damping</td>
<td>3</td>
</tr>
<tr>
<td>7.D.3R</td>
<td>Damping</td>
<td>3R</td>
</tr>
<tr>
<td>7.D.4</td>
<td>Damping</td>
<td>4</td>
</tr>
<tr>
<td>7.S.1</td>
<td>Seismic</td>
<td>1</td>
</tr>
<tr>
<td>7.S.2</td>
<td>Seismic</td>
<td>2</td>
</tr>
<tr>
<td>7.S.3</td>
<td>Seismic</td>
<td>3</td>
</tr>
<tr>
<td>7.S.3R</td>
<td>Seismic</td>
<td>3R</td>
</tr>
<tr>
<td>7.S.4R</td>
<td>Seismic</td>
<td>4</td>
</tr>
</tbody>
</table>

Note: An "R" followed by the test level denotes a repeat test of a particular test level.
Appendix B

Table 13.6: Test Phase 7A

<table>
<thead>
<tr>
<th>Test Designation</th>
<th>Test Type</th>
<th>Test Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>7A.F.2</td>
<td>Frequency</td>
<td>2</td>
</tr>
<tr>
<td>7A.F.3</td>
<td>Frequency</td>
<td>3</td>
</tr>
<tr>
<td>7A.F.4</td>
<td>Frequency</td>
<td>4</td>
</tr>
<tr>
<td>7A.D.2</td>
<td>Damping</td>
<td>2</td>
</tr>
<tr>
<td>7A.D.3</td>
<td>Damping</td>
<td>3</td>
</tr>
<tr>
<td>7A.D.4</td>
<td>Damping</td>
<td>4</td>
</tr>
<tr>
<td>7A.S.3</td>
<td>Seismic</td>
<td>3</td>
</tr>
<tr>
<td>7A.S.4</td>
<td>Seismic</td>
<td>4</td>
</tr>
</tbody>
</table>

Table 13.7: Test Phase 8

<table>
<thead>
<tr>
<th>Test Designation</th>
<th>Test Type</th>
<th>Test Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.Q</td>
<td>Quasi-Static</td>
<td>-</td>
</tr>
<tr>
<td>8.F.0</td>
<td>Frequency</td>
<td>0</td>
</tr>
<tr>
<td>8.F.1</td>
<td>Frequency</td>
<td>1</td>
</tr>
<tr>
<td>8.F.2</td>
<td>Frequency</td>
<td>2</td>
</tr>
<tr>
<td>8.F.3</td>
<td>Frequency</td>
<td>3</td>
</tr>
<tr>
<td>8.F.3R</td>
<td>Frequency</td>
<td>3R</td>
</tr>
<tr>
<td>8.F.4</td>
<td>Frequency</td>
<td>4</td>
</tr>
<tr>
<td>8.D.0</td>
<td>Damping</td>
<td>0</td>
</tr>
<tr>
<td>8.D.1</td>
<td>Damping</td>
<td>1</td>
</tr>
<tr>
<td>8.D.2</td>
<td>Damping</td>
<td>2</td>
</tr>
<tr>
<td>8.D.3</td>
<td>Damping</td>
<td>3</td>
</tr>
<tr>
<td>8.D.3R</td>
<td>Damping</td>
<td>3R</td>
</tr>
<tr>
<td>8.D.4</td>
<td>Damping</td>
<td>4</td>
</tr>
<tr>
<td>8.S.1</td>
<td>Seismic</td>
<td>1</td>
</tr>
<tr>
<td>8.S.2</td>
<td>Seismic</td>
<td>2</td>
</tr>
<tr>
<td>8.S.3</td>
<td>Seismic</td>
<td>3</td>
</tr>
<tr>
<td>8.S.3R</td>
<td>Seismic</td>
<td>3R</td>
</tr>
<tr>
<td>8.S.4</td>
<td>Seismic</td>
<td>4</td>
</tr>
</tbody>
</table>

Note: An "R" followed by the test level denotes a repeat test of a particular test level.
## Appendix B

Table 13.8: Test Phase 9

<table>
<thead>
<tr>
<th>Test Designation</th>
<th>Test Type</th>
<th>Test Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.Q</td>
<td>Quasi-Static</td>
<td>-</td>
</tr>
<tr>
<td>9.F.0</td>
<td>Frequency</td>
<td>0</td>
</tr>
<tr>
<td>9.F.1</td>
<td>Frequency</td>
<td>1</td>
</tr>
<tr>
<td>9.F.2</td>
<td>Frequency</td>
<td>2</td>
</tr>
<tr>
<td>9.F.3</td>
<td>Frequency</td>
<td>3</td>
</tr>
<tr>
<td>9.F.3R</td>
<td>Frequency</td>
<td>3R</td>
</tr>
<tr>
<td>9.F.4</td>
<td>Frequency</td>
<td>4</td>
</tr>
<tr>
<td>9.F.5</td>
<td>Frequency</td>
<td>5</td>
</tr>
<tr>
<td>9.D.0</td>
<td>Damping</td>
<td>0</td>
</tr>
<tr>
<td>9.D.1</td>
<td>Damping</td>
<td>1</td>
</tr>
<tr>
<td>9.D.2</td>
<td>Damping</td>
<td>2</td>
</tr>
<tr>
<td>9.D.3</td>
<td>Damping</td>
<td>3</td>
</tr>
<tr>
<td>9.D.3R</td>
<td>Damping</td>
<td>3R</td>
</tr>
<tr>
<td>9.D.4</td>
<td>Damping</td>
<td>4</td>
</tr>
<tr>
<td>9.D.5</td>
<td>Damping</td>
<td>5</td>
</tr>
<tr>
<td>9.S.1</td>
<td>Seismic</td>
<td>1</td>
</tr>
<tr>
<td>9.S.2</td>
<td>Seismic</td>
<td>2</td>
</tr>
<tr>
<td>9.S.3</td>
<td>Seismic</td>
<td>3</td>
</tr>
<tr>
<td>9.S.3R</td>
<td>Seismic</td>
<td>3R</td>
</tr>
<tr>
<td>9.S.4R</td>
<td>Seismic</td>
<td>4</td>
</tr>
<tr>
<td>9.S.5</td>
<td>Seismic</td>
<td>5</td>
</tr>
</tbody>
</table>

Note: An "R" followed by the test level denotes a repeat test of a particular test level.
Table 13.9: Test Phase 10

<table>
<thead>
<tr>
<th>Test Designation</th>
<th>Test Type</th>
<th>Test Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.Q</td>
<td>Quasi-Static</td>
<td>-</td>
</tr>
<tr>
<td>10.F.0</td>
<td>Frequency</td>
<td>0</td>
</tr>
<tr>
<td>10.F.1</td>
<td>Frequency</td>
<td>1</td>
</tr>
<tr>
<td>10.F.2</td>
<td>Frequency</td>
<td>2</td>
</tr>
<tr>
<td>10.F.3</td>
<td>Frequency</td>
<td>3</td>
</tr>
<tr>
<td>10.F.3R</td>
<td>Frequency</td>
<td>3R</td>
</tr>
<tr>
<td>10.F.4</td>
<td>Frequency</td>
<td>4</td>
</tr>
<tr>
<td>10.F.5</td>
<td>Frequency</td>
<td>5</td>
</tr>
<tr>
<td>10.D.0</td>
<td>Damping</td>
<td>0</td>
</tr>
<tr>
<td>10.D.1</td>
<td>Damping</td>
<td>1</td>
</tr>
<tr>
<td>10.D.2</td>
<td>Damping</td>
<td>2</td>
</tr>
<tr>
<td>10.D.3</td>
<td>Damping</td>
<td>3</td>
</tr>
<tr>
<td>10.D.3R</td>
<td>Damping</td>
<td>3R</td>
</tr>
<tr>
<td>10.D.4</td>
<td>Damping</td>
<td>4</td>
</tr>
<tr>
<td>10.D.5</td>
<td>Damping</td>
<td>5</td>
</tr>
<tr>
<td>10.S.1</td>
<td>Seismic</td>
<td>1</td>
</tr>
<tr>
<td>10.S.2</td>
<td>Seismic</td>
<td>2</td>
</tr>
<tr>
<td>10.S.3</td>
<td>Seismic</td>
<td>3</td>
</tr>
<tr>
<td>10.S.3R</td>
<td>Seismic</td>
<td>3R</td>
</tr>
<tr>
<td>10.S.4R</td>
<td>Seismic</td>
<td>4</td>
</tr>
<tr>
<td>10.S.5</td>
<td>Seismic</td>
<td>5</td>
</tr>
<tr>
<td>10.S.5R</td>
<td>Seismic</td>
<td>5R</td>
</tr>
</tbody>
</table>

Note: An "R" followed by the test level denotes a repeat test of a particular test level.
Appendix B

Figure 13.1: Typical View of Supplemental Weights on 2nd Story Walls and Floor Diaphragm
Appendix B

Figure 13.2: Typical View of Supplemental Weights on Roof Trusses

Figure 13.3: Elevation View of Supplemental Weights on 2nd Story East and West Walls for Test Phase 5

Note: Number on block indicates number of blocks in stack. Each block on walls weighs 72 lbs.
Appendix B

Figure 13.4: Elevation View of Supplemental Weights on 2nd Story East and West Walls for Test Phases 6 – 9

Figure 13.5: Elevation View of Supplemental Weights on 2nd Story North and South Walls for Test Phases 5 – 9
Appendix B

Figure 13.6: Plan View of Supplemental Weights on 2nd Floor for Test Phases 5 – 9

Note: Number on block indicates number of blocks in stack. Each block on floor weighs 52 lbs.

Note: Each block attached to roof trusses weighs 21 lbs.
Figure 13.7: Plan View of Supplemental Weights in Roof Trusses for Test Phases 5 – 9
Appendix B

Figure 13.8: Plan View of Supplemental Weights in Roof Trusses for Test Phase 10
Appendix C  Instrumentation
### Table 13.10: Summary of Instrument Locations, Data Units, and Sign Conventions

<table>
<thead>
<tr>
<th>Channel</th>
<th>Measurement</th>
<th>Location</th>
<th>Data Units</th>
<th>Sign Convention</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>Shake Table Absolute Displacement, North-South Direction</td>
<td>Make Shake, South Wall</td>
<td>Inches</td>
<td>Absolute Displacement to the North in Positive</td>
</tr>
<tr>
<td>B1</td>
<td>Shake Table Absolute Acceleration, North-South Direction</td>
<td>Make Shake, South Wall</td>
<td>g's</td>
<td>Absolute Acceleration to the North in Positive</td>
</tr>
<tr>
<td>C1 - C10</td>
<td>Test Structure Absolute Displacement, North-South Direction</td>
<td>South Side of Test Structure &amp; Roof</td>
<td>Inches</td>
<td>Absolute Displacement to the North in Positive</td>
</tr>
<tr>
<td>D1 - D23</td>
<td>Test Structure Absolute Acceleration, North-South Direction</td>
<td>South Side of Test Structure &amp; Roof</td>
<td>g's</td>
<td>Absolute Acceleration to the North in Positive</td>
</tr>
<tr>
<td>E1 - E12</td>
<td>Test Structure Absolute Acceleration, East-West Direction</td>
<td>East Side of Test Structure</td>
<td>g's</td>
<td>Absolute Acceleration to the East in Positive</td>
</tr>
<tr>
<td>F1 - F5</td>
<td>North Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F6 - F10</td>
<td>South Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F11 - F22</td>
<td>East Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F23 - F30</td>
<td>West Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F31 - F33</td>
<td>Interior Bearing Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H1 - H2</td>
<td>1st Story North Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H3 - H10</td>
<td>2nd Story North Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H11 - H12</td>
<td>1st Story South Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H13 - H20</td>
<td>2nd Story South Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H21 - H24</td>
<td>1st Story East Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H25 - H28</td>
<td>2nd Story East Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H29 - H32</td>
<td>1st Story West Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H33 - H36</td>
<td>2nd Story West Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H37 - H40</td>
<td>1st Story East Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H41 - H44</td>
<td>2nd Story East Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H45 - H48</td>
<td>1st Story West Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H49 - H52</td>
<td>2nd Story West Shear Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>K1 - K2</td>
<td>1st Story North Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>K3 - K10</td>
<td>2nd Story North Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>K11 - K12</td>
<td>1st Story South Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>K13 - K20</td>
<td>2nd Story South Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>K21 - K24</td>
<td>1st Story East Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>K25 - K28</td>
<td>2nd Story East Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>K29 - K32</td>
<td>1st Story West Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>K33 - K36</td>
<td>2nd Story West Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>L1 - L14</td>
<td>Diagonal Deformation (Racking) of 2nd Floor Diaphragm</td>
<td>2nd Floor Diaphragm</td>
<td>Inches</td>
<td>Extension in Positive</td>
</tr>
<tr>
<td>M1 - M5</td>
<td>Chord at North Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>M6 - M10</td>
<td>Chord at South Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>N1 - N11</td>
<td>Second Floor Diaphragm Absolute Displacement</td>
<td>2nd Floor Diaphragm</td>
<td>Inches</td>
<td>Extension in Positive</td>
</tr>
<tr>
<td>O1 - O10</td>
<td>Second Floor Diaphragm Absolute Acceleration</td>
<td>2nd Floor Diaphragm</td>
<td>g's</td>
<td>Absolute Acceleration to the North in Positive</td>
</tr>
<tr>
<td>P1 - P9</td>
<td>Roof Diaphragm Absolute Displacement</td>
<td>Roof Diaphragm</td>
<td>Inches</td>
<td>Absolute Displacement to the North in Positive</td>
</tr>
<tr>
<td>P10 - P14</td>
<td>Roof Diaphragm Absolute Acceleration</td>
<td>Roof Diaphragm</td>
<td>g's</td>
<td>Absolute Acceleration to the North in Positive</td>
</tr>
<tr>
<td>Q1 - Q2</td>
<td>1st Story North Wall Sill Plate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Q3 - Q4</td>
<td>1st Story South Wall Sill Plate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Q5 - Q6</td>
<td>1st Story East Wall Sill Plate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Q7 - Q8</td>
<td>1st Story West Wall Sill Plate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Q9 - Q10</td>
<td>2nd Story North Wall Sill Plate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Q11 - Q12</td>
<td>2nd Story South Wall Sill Plate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Q13 - Q14</td>
<td>2nd Story East Wall Sill Plate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Q15 - Q16</td>
<td>2nd Story West Wall Sill Plate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>R1 - R9</td>
<td>Base Absolute Displacement, North-South Direction</td>
<td>South Beam of Base</td>
<td>Inches</td>
<td>Absolute Displacement to the North in Positive</td>
</tr>
<tr>
<td>S1 - S3</td>
<td>Base Absolute Acceleration, North-South Direction</td>
<td>South Beam of Base</td>
<td>g's</td>
<td>Absolute Acceleration to the North in Positive</td>
</tr>
<tr>
<td>S4 - S5</td>
<td>Base Absolute Acceleration, East-West Direction</td>
<td>East Beam of Base</td>
<td>g's</td>
<td>Absolute Acceleration to the East in Positive</td>
</tr>
<tr>
<td>T1 - T10</td>
<td>Test Structure Relative Displacement, North-South Direction</td>
<td>North Wall</td>
<td>Inches</td>
<td>Relative Displacement to the North in Positive</td>
</tr>
<tr>
<td>U1 - U2</td>
<td>Axial Load in Quasi-Static Loading Arm</td>
<td>South Wall</td>
<td>Kips</td>
<td>Tension in Positive (Equivalent to a Positive Concentrated Load Pinning South)</td>
</tr>
<tr>
<td>V1 - V2</td>
<td>1st Story North Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>V3 - V10</td>
<td>2nd Story North Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>V11 - V12</td>
<td>1st Story South Wall</td>
<td></td>
<td>Inches</td>
<td>Absolute Displacement to the North in Positive</td>
</tr>
<tr>
<td>V13 - V20</td>
<td>2nd Story South Wall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>V21 - V23</td>
<td>1st Story East Wall</td>
<td></td>
<td>Inches</td>
<td>Absolute Displacement to the North in Positive</td>
</tr>
<tr>
<td>V24 - V26</td>
<td>1st Story West Wall</td>
<td></td>
<td>Inches</td>
<td>Absolute Displacement to the North in Positive</td>
</tr>
<tr>
<td>V27 - V30</td>
<td>2nd Story East Wall</td>
<td></td>
<td>Inches</td>
<td>Absolute Displacement to the North in Positive</td>
</tr>
<tr>
<td>V31 - V36</td>
<td>2nd Story West Wall</td>
<td></td>
<td>Inches</td>
<td>Absolute Displacement to the North in Positive</td>
</tr>
</tbody>
</table>
Figure 13.9: Instrumentation Plan for Test Phases 1 – 4 (North-East View)
Figure 13.10: Instrumentation Plan for Test Phases 1 – 4 (South-West View)
Appendix C

Figure 13.11: Instrumentation Plan for Test Phases 1 – 4 (2nd Floor View)
Figure 13.12: Instrumentation Plan for Test Phase 5 (North-East View)
Appendix C

Figure 13.13: Instrumentation Plan for Test Phase 5 (South-West View)
Figure 13.14: Instrumentation Plan for Test Phase 5 (2\textsuperscript{nd} Floor View)
Figure 13.15: Instrumentation Plan for Test Phases 6 & 6A (North-East View)
Figure 13.16: Instrumentation Plan for Test Phases 6 & 6A (South-West View)
Figure 13.17: Instrumentation Plan for Test Phases 6 & 6A (2nd Floor View)
Figure 13.18: Instrumentation Plan for Test Phase 7 (North-East View)
Figure 13.19: Instrumentation Plan for Test Phase 7 (South-West View)
Figure 13.20: Instrumentation Plan for Test Phase 7 (2nd Floor View)
Figure 13.21: Instrumentation Plan for Test Phase 7A (North-East View)
Figure 13.22: Instrumentation Plan for Test Phase 7A (South-West View)
Appendix C

Figure 13.23: Instrumentation Plan for Test Phase 7A (2nd Floor View)
Appendix C

Figure 13.24: Instrumentation Plan for Test Phase 8 (North-East View)
Figure 13.25: Instrumentation Plan for Test Phase 8 (South-West View)
Appendix C

Figure 13.26: Instrumentation Plan for Test Phase 8 (2nd Floor View)
Figure 13.27: Instrumentation Plan for Test Phase 9 (North-East View)
Figure 13.28: Instrumentation Plan for Test Phase 9 (South-West View)
Appendix C

Figure 13.29: Instrumentation Plan for Test Phase 9 (2nd Floor View)
Appendix C

Figure 13.30: Instrumentation Plan for Test Phase 10 (North-East View)
Figure 13.31: Instrumentation Plan for Test Phase 10 (South-West View)
Appendix C

Figure 13.32: Instrumentation Plan for Test Phase 10 (2nd Floor View)
### Appendix C

Table 13.11: Lengths of Racking/Chord Deformation Instruments for Test Phases 1 - 4

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Initial Length (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Diagonal Component</td>
</tr>
<tr>
<td>H21 &amp; H24</td>
<td>211.76</td>
</tr>
<tr>
<td>H29 &amp; H32</td>
<td>211.76</td>
</tr>
<tr>
<td>H37 &amp; H40</td>
<td>93.50</td>
</tr>
<tr>
<td>H45 &amp; H48</td>
<td>93.50</td>
</tr>
<tr>
<td>L1-L2</td>
<td>196.28</td>
</tr>
<tr>
<td>L3-L4</td>
<td>196.28</td>
</tr>
<tr>
<td>L5-L6</td>
<td>196.28</td>
</tr>
<tr>
<td>L7-L8</td>
<td>196.28</td>
</tr>
<tr>
<td>L9-L10</td>
<td>196.28</td>
</tr>
<tr>
<td>L11-L14</td>
<td>213.02</td>
</tr>
<tr>
<td>M1-M10</td>
<td>47.50</td>
</tr>
</tbody>
</table>

Table 13.12: Lengths of Racking/Chord Deformation Instruments for Test Phase 5

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Initial Length (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Diagonal Component</td>
</tr>
<tr>
<td>H1-H2</td>
<td>169.25</td>
</tr>
<tr>
<td>H3-H10</td>
<td>96.24</td>
</tr>
<tr>
<td>H11-H12</td>
<td>169.25</td>
</tr>
<tr>
<td>H13-H20</td>
<td>96.24</td>
</tr>
<tr>
<td>H21 &amp; H24</td>
<td>211.76</td>
</tr>
<tr>
<td>H25 &amp; H28</td>
<td>211.76</td>
</tr>
<tr>
<td>H29 &amp; H32</td>
<td>211.76</td>
</tr>
<tr>
<td>H33 &amp; H36</td>
<td>211.76</td>
</tr>
<tr>
<td>H37 &amp; H40</td>
<td>93.50</td>
</tr>
<tr>
<td>H41 &amp; H44</td>
<td>93.50</td>
</tr>
<tr>
<td>H45 &amp; H48</td>
<td>93.50</td>
</tr>
<tr>
<td>H49 &amp; H52</td>
<td>93.50</td>
</tr>
<tr>
<td>L1-L2</td>
<td>193.27</td>
</tr>
<tr>
<td>L3-L4</td>
<td>194.54</td>
</tr>
<tr>
<td>L5-L6</td>
<td>193.99</td>
</tr>
<tr>
<td>L7-L8</td>
<td>194.54</td>
</tr>
<tr>
<td>L9-L10</td>
<td>193.27</td>
</tr>
<tr>
<td>M1-M10</td>
<td>47.63</td>
</tr>
</tbody>
</table>
Table 13.13: Lengths of Racking/Chord Deformation Instruments for Test Phases 6 - 8

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Initial Length (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Diagonal Component</td>
</tr>
<tr>
<td>H1-H2</td>
<td>169.25</td>
</tr>
<tr>
<td>H3-H10</td>
<td>96.24</td>
</tr>
<tr>
<td>H11-H12</td>
<td>169.25</td>
</tr>
<tr>
<td>H13-H20</td>
<td>96.24</td>
</tr>
<tr>
<td>H21-H24</td>
<td>119.71</td>
</tr>
<tr>
<td>H25-H28</td>
<td>100.19</td>
</tr>
<tr>
<td>H29-H32</td>
<td>119.71</td>
</tr>
<tr>
<td>H33-H36</td>
<td>100.19</td>
</tr>
<tr>
<td>H37-H40</td>
<td>93.50</td>
</tr>
<tr>
<td>H41-H44</td>
<td>93.50</td>
</tr>
<tr>
<td>H45-H48</td>
<td>93.50</td>
</tr>
<tr>
<td>H49-H52</td>
<td>93.50</td>
</tr>
<tr>
<td>L1-L2</td>
<td>193.27</td>
</tr>
<tr>
<td>L3-L4</td>
<td>194.54</td>
</tr>
<tr>
<td>L5-L6</td>
<td>193.99</td>
</tr>
<tr>
<td>L7-L8</td>
<td>194.54</td>
</tr>
<tr>
<td>L9-L10</td>
<td>193.27</td>
</tr>
<tr>
<td>M1-M10</td>
<td>47.63</td>
</tr>
</tbody>
</table>
## Appendix C

Table 13.14: Lengths of Racking/Chord Deformation Instruments for Test Phase 9

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Initial Length (inches)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Measurement</td>
<td>Diagonal Component</td>
<td>Horizontal Component</td>
</tr>
<tr>
<td>H1-H2</td>
<td>169.25</td>
<td>140.75</td>
</tr>
<tr>
<td>H3-H10</td>
<td>96.24</td>
<td>34.75</td>
</tr>
<tr>
<td>H11-H12</td>
<td>169.25</td>
<td>140.75</td>
</tr>
<tr>
<td>H13-H20</td>
<td>96.24</td>
<td>34.75</td>
</tr>
<tr>
<td>H21-H24</td>
<td>99.40</td>
<td>33.75</td>
</tr>
<tr>
<td>H25-H28</td>
<td>100.19</td>
<td>36.00</td>
</tr>
<tr>
<td>H29-H32</td>
<td>119.71</td>
<td>74.75</td>
</tr>
<tr>
<td>H33-H36</td>
<td>100.19</td>
<td>36.00</td>
</tr>
<tr>
<td>H37-H40</td>
<td>93.50</td>
<td>0.00</td>
</tr>
<tr>
<td>H41-H44</td>
<td>93.50</td>
<td>0.00</td>
</tr>
<tr>
<td>H45-H48</td>
<td>93.50</td>
<td>0.00</td>
</tr>
<tr>
<td>H49-H52</td>
<td>93.50</td>
<td>0.00</td>
</tr>
<tr>
<td>L1-L2</td>
<td>193.27</td>
<td>-</td>
</tr>
<tr>
<td>L3-L4</td>
<td>194.54</td>
<td>-</td>
</tr>
<tr>
<td>L5-L6</td>
<td>193.99</td>
<td>-</td>
</tr>
<tr>
<td>L7-L8</td>
<td>194.54</td>
<td>-</td>
</tr>
<tr>
<td>L9-L10</td>
<td>193.27</td>
<td>-</td>
</tr>
<tr>
<td>M1-M10</td>
<td>47.63</td>
<td>47.63</td>
</tr>
</tbody>
</table>

Table 13.15: Lengths of Racking/Chord Deformation Instruments for Test Phase 10

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Initial Length (inches)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Measurement</td>
<td>Diagonal Component</td>
<td>Horizontal Component</td>
</tr>
<tr>
<td>H1-H2</td>
<td>169.25</td>
<td>140.75</td>
</tr>
<tr>
<td>H3-H10</td>
<td>96.24</td>
<td>34.75</td>
</tr>
<tr>
<td>H11-H12</td>
<td>169.25</td>
<td>140.75</td>
</tr>
<tr>
<td>H13-H20</td>
<td>96.24</td>
<td>34.75</td>
</tr>
<tr>
<td>H21-H24</td>
<td>99.40</td>
<td>33.75</td>
</tr>
<tr>
<td>H25-H28</td>
<td>100.19</td>
<td>36.00</td>
</tr>
<tr>
<td>H29-H32</td>
<td>119.71</td>
<td>74.75</td>
</tr>
<tr>
<td>H33-H36</td>
<td>100.19</td>
<td>36.00</td>
</tr>
<tr>
<td>H37-H40</td>
<td>93.50</td>
<td>0.00</td>
</tr>
<tr>
<td>H41-H44</td>
<td>93.50</td>
<td>0.00</td>
</tr>
<tr>
<td>H45-H48</td>
<td>93.50</td>
<td>0.00</td>
</tr>
<tr>
<td>H49-H52</td>
<td>93.50</td>
<td>0.00</td>
</tr>
<tr>
<td>M1-M10</td>
<td>47.63</td>
<td>47.63</td>
</tr>
</tbody>
</table>
Appendix D  Sheathing to Framing Lumber Connection Test Results
Appendix D

OSB - Parallel (Monotonic) Connection Tests

Configuration: Wall sheathing to studs
Loading: Monotonic
Direction: Parallel to grain
Sheathing: 3/8” OSB
Nailing: 8d box gun nails (load data represents one nail connector)
Appendix D
OSB - Parallel (Cyclic)

Configuration: Wall sheathing to studs
Loading: Cyclic
Direction: Parallel to grain
Sheathing: 3/8” OSB
Nailing: 8d box gun nails (load data represents one nail connector)
Delta: 0.65”
Appendix D

OSB - Perpendicular (Monotonic) Connection Tests

Configuration: Wall sheathing to studs
Loading: Monotonic
Direction: Perpendicular to grain
Sheathing: 3/8” OSB
Nailing: 8d box gun nails (load data represents one nail connector)
Appendix D

OSB - Perpendicular (Cyclic) Connection Tests

Configuration: Wall sheathing to studs
Loading: Cyclic
Direction: Perpendicular to grain
Sheathing: 3/8" OSB
Nailing: 8d box gun nails (load data represents one nail connector)
Delta: 0.65"

Specimen 1
Specimen 2
Specimen 3
Appendix D

Plywood - Parallel (Monotonic) Connection Tests

Configuration: Floor sheathing to joists
Loading: Monotonic
Direction: Parallel to grain
Sheathing: 3/4" PLY
Nailing: 10d gun nails (0.131" diameter) (load data represents one nail connector)
Appendix E  Selected Quasi-Static Results
Appendix E
Quasi-Static Test 1.Q

One story, east & west fully sheathed, north & south framed with openings, second floor diaphragm nailed at 50% with no blocking & no adhesive.

Overall Stiffness,
\[ k = 73.5 \text{ kips/in} \]

Equivalent Viscous Damping Ratio,
\[ \zeta_{eq} = 22.5\% \]

Equivalent Shear Stiffness,
\[ G A_s = 6,657 \text{ kips} \]

Equivalent Flexural Stiffness,
\[ EI = 56.0 \times 10^6 \text{ kip-in}^2 \]
Appendix E

Quasi-Static Test 2.Q

One story, east & west fully sheathed, north & south framed with openings, second floor diaphragm nailed at 100% with no blocking & no adhesive.

Global Diaphragm Hysteresis Loop

Overall Stiffness,

\[ k = 86.0 \text{ kips/in} \]

Equivalent Viscous Damping Ratio,

\[ \zeta_{eq} = 19.1\% \]

Diaphragm Shear Hysteresis Loop

Equivalent Shear Stiffness,

\[ GA_s = 7,143 \text{ kips} \]

Diaphragm Flexural Hysteresis Loop

Equivalent Flexural Stiffness,

\[ EI = 69.8 \times 10^6 \text{ kip-in}^2 \]

Diaphragm Deformation Hysteresis Loop
Appendix E

Quasi-Static Test 3A.Q

One story, east & west fully sheathed, north & south framed with openings, second floor diaphragm nailed at 100% with 2 x 10 blocking & no adhesive.

Global Diaphragm Hysteresis Loop

Overall Stiffness,
\[ k = 109.3 \text{ kips/in} \]

Equivalent Viscous Damping Ratio,
\[ \zeta_{eq} = 13.5\% \]

Equivalent Shear Stiffness,
\[ GA_s = 19,708 \text{ kips} \]

Equivalent Flexural Stiffness,
\[ EI = 45.3 \times 10^6 \text{ kip-in}^2 \]

Diaphragm Shear Hysteresis Loop

Diaphragm Flexural Hysteresis Loop

Diaphragm Deformation Hysteresis Loop
Appendix E
Quasi-Static Test 3B.Q

One story, east & west fully sheathed, north & south framed with openings, second floor diaphragm nailed at 100% with 3 x 4 blocking & no adhesive.

Global Diaphragm Hysteresis Loop

Overall Stiffness,
\[ k = 114.6 \text{ kips/in} \]
Equivalent Viscous Damping Ratio,
\[ \zeta_{eq} = 13.7\% \]

Diaphragm Shear Hysteresis Loop

Equivalent Shear Stiffness,
\[ GA_s = 32,322 \text{ kips} \]

Diaphragm Flexural Hysteresis Loop

Equivalent Flexural Stiffness,
\[ EI = 40.5 \times 10^6 \text{ kip-in}^2 \]

Diaphragm Deformation Hysteresis Loop
Appendix E
Quasi-Static Test 4A.Q

One story, east & west fully sheathed, north & south framed with openings, second floor diaphragm nailed at 50% with no blocking & PL400 adhesive.

Overall Stiffness,

\[ k = 105.3 \text{ kips/in} \]

Equivalent Viscous Damping Ratio,

\[ \zeta_{eq} = 13.6\% \]

Equivalent Shear Stiffness,

\[ G\alpha_s = 24,354 \text{ kips} \]

Equivalent Flexural Stiffness,

\[ EI = 40.0 \times 10^6 \text{ kip-in}^2 \]
Appendix E

Quasi-Static Test 4B.Q

One story, east & west fully sheathed, north & south framed with openings, second floor diaphragm nailed at 100% with 3 x 4 blocking & PL400 adhesive.

Overall Stiffness,
\[ k = 110.1 \text{ kips/in} \]

Equivalent Viscous Damping Ratio,
\[ \zeta_{eq} = 12.6\% \]

Equivalent Shear Stiffness,
\[ G\alpha_s = 46,071 \text{ kips} \]

Equivalent Flexural Stiffness,
\[ EI = 36.2 \times 10^6 \text{ kip-in}^2 \]
Appendix E

Quasi-Static Test 4C.Q

One story, east & west fully sheathed, north & south framed with openings, second floor diaphragm nailed at 100% with 2 x 10 blocking & PL400 adhesive.

Overall Stiffness,

\[ k = 134.7 \text{ kips/in} \]

Equivalent Viscous Damping Ratio,

\[ \zeta_{eq} = 12.5\% \]

Equivalent Shear Stiffness,

\[ GA_s = 51,408 \text{ kips} \]

Equivalent Flexural Stiffness,

\[ EI = 44.7 \times 10^6 \text{ kip-in}^2 \]
Appendix E

Quasi-Static Test 4D.Q

One story, east & west fully sheathed, north & south framed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Overall Stiffness,
\[ k = 101.7 \text{ kips/in} \]

Equivalent Viscous Damping Ratio,
\[ \zeta_{eq} = 13.6\% \]

Equivalent Shear Stiffness,
\[ GA_s = 31,580 \text{ kips} \]

Equivalent Flexural Stiffness,
\[ EI = 35.8 \times 10^6 \text{ kip-in}^2 \]
Appendix E

Quasi-Static Test 5.Q

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Global Stiffness,
\[ k = 296.6 \text{ kips/in} \]

Equivalent Viscous Damping Ratio,
\[ \zeta_{eq} = 5.6\% \]

Equivalent Shear Stiffness,
\[ GA_s = 36,450 \text{ kips} \]

Equivalent Flexural Stiffness,
\[ EI = 156.6 \times 10^6 \text{ kip-in}^2 \]
Appendix E

Quasi-Static Test 6.Q

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Overall Stiffness,
\[ k = 277.6 \text{ kips/in} \]

Equivalent Viscous Damping Ratio,
\[ \zeta_{eq} = 6.0\% \]

Equivalent Shear Stiffness,
\[ G A_s = 30,225 \text{ kips} \]

Equivalent Flexural Stiffness,
\[ EI = 157.8 \times 10^6 \text{ kip-in}^2 \]
Appendix E

Quasi-Static Test 7.Q

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Overall Stiffness,
\[ k = 238.3 \text{ kips/in} \]

Equivalent Viscous Damping Ratio,
\[ \zeta_{eq} = 8.1\% \]

Equivalent Shear Stiffness,
\[ GA_s = 87,992 \text{ kips} \]

Equivalent Flexural Stiffness,
\[ EI = 80.8 \times 10^6 \text{ kip-in}^2 \]
Appendix E

Quasi-Static Test 8.Q

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Overall Stiffness,
\[ k = 217.1 \text{ kips/in} \]

Equivalent Viscous Damping Ratio,
\[ \zeta_{eq} = 6.6\% \]

Equivalent Shear Stiffness,
\[ GA_s = 31,283 \text{ kips} \]

Equivalent Flexural Stiffness,
\[ EI = 102.8 \times 10^6 \text{ kip-in}^2 \]
Appendix E
Quasi-Static Test 9.Q

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Overall Stiffness,
\[ k = 216.6 \text{ kips/in} \]

Equivalent Viscous Damping Ratio,
\[ \zeta_{eq} = 8.3\% \]

Equivalent Shear Stiffness,
\[ G\alpha_s = 40,928 \text{ kips} \]

Equivalent Flexural Stiffness,
\[ EI = 88.6 \times 10^6 \text{ kip-in}^2 \]
Appendix E

Quasi-Static Test 10.Q

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Overall Stiffness,
\[ k = 425.7 \text{ kips/in} \]

Equivalent Viscous Damping Ratio,
\[ \zeta_{eq} = 10.1\% \]

No Shear Deformation Data Acquired
Appendix F  Selected Frequency Test Results
Appendix F
Frequency Test 5.F.0

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL-400 adhesive.

Fundamental Mode, Frequency = 5.62 Hz

Spectral Densities

Channel D23  
Channel D18  
Channel D8

Frequency (Hz)  
Frequency (Hz)  
Frequency (Hz)

Phase Angles

Channel D18 - Channel D23  
Channel D8 - Channel D23

Channel D19 - Channel D23  
Channel D14 - Channel D23  
Channel D4 - Channel D23

Phase (degrees)  
Phase (degrees)  
Phase (degrees)

Frequency (Hz)  
Frequency (Hz)  
Frequency (Hz)
Appendix F

Frequency Test 5.F.1

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL-400 adhesive.

Fundamental Mode, Frequency = 5.42 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 5.F.2

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 5.37 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 5.F.3

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 5.37 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 5.F.4

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL-400 adhesive.

Fundamental Mode, Frequency = 5.13 Hz

Spectral Densities

Phase Angles
Appendix F
Frequency Test 5.F.4R

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode

$\nu_f = 4.98$ Hz

Spectral Densities

Phase Angles

Channel D1 - Channel D23
Channel D8 - Channel D23
Channel D19 - Channel D23
Channel D14 - Channel D23
Channel D4 - Channel D23
Appendix F

Frequency Test 6.F.0

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 4.25 Hz

Spectral Densities

Phase Angles
Appendix F
Frequency Test 6.F.1

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 4.25 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 6.F.2

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 4.05 Hz

Spectral Densities

Phase Angles
Appendix F
Frequency Test 6.F.3

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 4.05 Hz

Spectral Densities

Phase Angles
Appendix F
Frequency Test 6.F.4

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.66 Hz

Spectral Densities

Phase Angles
Appendix F
Frequency Test 6.F.4R

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.71 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 6A.F.2

Two stories with roof, east & west sheathed with window and small door openings with "wastewall" sheathing removed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.27 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 6A.F.3

Two stories with roof, east & west sheathed with window and small door openings with "wastewall" sheathing removed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.22 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 7.F.0

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.91 Hz

Spectral Densities

Phase Angles

Channel D18 - Channel D23

Channel D8 - Channel D23

Channel D19 - Channel D23

Channel D14 - Channel D23

Channel D4 - Channel D23
Appendix F
Frequency Test 7.F.1

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.91 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 7.F.2

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.91 Hz

Spectral Densities

Phase Angles
Appendix F
Frequency Test 7.F.3

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.66 Hz

Spectral Densities

Phase Angles
Appendix F
Frequency Test 7.F.3R
Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

**Fundamental Mode, Frequency = 3.56 Hz**

**Spectral Densities**

**Phase Angles**

---

Channel D18 - Channel D23

Channel D19 - Channel D23

Channel D14 - Channel D23

Channel D4 - Channel D23
Appendix F

Frequency Test 7.F.4

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.17 Hz

Spectral Densities

Phase Angles
Appendix F
Frequency Test 7A.F.2

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design with anchor bolt adjacent to door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.37 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 7A.F.3

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design with anchor bolt adjacent to door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.37 Hz

Spectral Densities

Phase Angles
Appendix F
Frequency Test 7A.F.4

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design with anchor bolt adjacent to door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.37 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 8.F.0

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 4.15 Hz

![Diagram of two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.]

Spectral Densities

<table>
<thead>
<tr>
<th>Channel</th>
<th>Amplitude (g-s)^2</th>
</tr>
</thead>
<tbody>
<tr>
<td>D23</td>
<td></td>
</tr>
<tr>
<td>D18</td>
<td></td>
</tr>
<tr>
<td>Channel</td>
<td>Amplitude (g-s)^2</td>
</tr>
<tr>
<td>D19</td>
<td></td>
</tr>
<tr>
<td>D14</td>
<td></td>
</tr>
<tr>
<td>Channel</td>
<td>Amplitude (g-s)^2</td>
</tr>
<tr>
<td>D4</td>
<td></td>
</tr>
</tbody>
</table>

Phase Angles

<table>
<thead>
<tr>
<th>Channel</th>
<th>Phase (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D18 - D23</td>
<td></td>
</tr>
<tr>
<td>D8 - D23</td>
<td></td>
</tr>
<tr>
<td>Channel</td>
<td>Phase (degrees)</td>
</tr>
<tr>
<td>D19 - D23</td>
<td></td>
</tr>
<tr>
<td>D14 - D23</td>
<td></td>
</tr>
<tr>
<td>Channel</td>
<td>Phase (degrees)</td>
</tr>
<tr>
<td>D4 - D23</td>
<td></td>
</tr>
</tbody>
</table>
Appendix F
Frequency Test 8.F.1

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 4.05 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 8.F.2

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 4.05 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 8.F.3

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.91 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 8.F.3R

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.81 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 8.F.4

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

**Fundamental Mode, Frequency = 3.47 Hz**

*East Elevation*

*West Elevation*

**Spectral Densities**

**Phase Angles**
Appendix F
Frequency Test 9.F.0

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.96 Hz

Spectral Densities

Phase Angles
Appendix F
Frequency Test 9.F.1

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.91 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 9.F.2

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.71 Hz

Spectral Densities

Phase Angles
Appendix F
Frequency Test 9.F.3

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.66 Hz

Spectral Densities

Phase Angles
Appendix F
Frequency Test 9.F.3R

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 3.42 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 9.F.4

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 2.93 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 9.F.5

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Fundamental Mode, Frequency = 2.93 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 10.F.0

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Fundamental Mode, Frequency = 6.49 Hz

Spectral Densities

Phase Angles
Appendix F
Frequency Test 10.F.1

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Fundamental Mode, Frequency = 6.35 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 10.F.2

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Fundamental Mode, Frequency = 6.10 Hz

Spectral Densities

Phase Angles
Appendix F
 Frequency Test 10.F.3

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Fundamental Mode, Frequency = 5.76 Hz

Spectral Densities

Phase Angles
Appendix F

Frequency Test 10.F.4

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Fundamental Mode, Frequency = 5.71 Hz

Spectral Densities

Phase Angles

Channel D23 - Channel D28
Channel D8 - Channel D23
Channel D19 - Channel D23
Channel D14 - Channel D23
Channel D4 - Channel D23
Appendix F

Frequency Test 10.F.4R

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Fundamental Mode, Frequency = 5.47 Hz

Spectral Densities

Phase Angles
Appendix F
Frequency Test 10.F.5
Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Fundamental Mode, Frequency = 5.37 Hz

Spectral Densities

Phase Angles
Appendix G  Selected Damping Test Results
Appendix G

Damping Test 5.D.0

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 5.62 Hz)

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>4.7</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>5.2</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>5.5</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 4.7%
Appendix G

Damping Test 5.D.1

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 5.42 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>4.5</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>4.0</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>4.0</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 5.7%
Appendix G

Damping Test 5.D.2

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping</td>
<td>Channel</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(% Critical)</td>
<td></td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>6.3</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>18.3</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>18.7</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 12.8%
Appendix G
Damping Test 5.D.3

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 5.37 Hz)

<table>
<thead>
<tr>
<th>Location</th>
<th>Channel</th>
<th>Damping (%)</th>
<th>Channel</th>
<th>Damping (%)</th>
<th>Channel</th>
<th>Damping (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>7.6</td>
<td>D21</td>
<td>11.6</td>
<td>D23</td>
<td>11.0</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>6.9</td>
<td>D16</td>
<td>7.6</td>
<td>D18</td>
<td>14.7</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>9.2</td>
<td>D6</td>
<td>8.0</td>
<td>D8</td>
<td>9.4</td>
</tr>
</tbody>
</table>

Mean Damping = 9.6%
Appendix G

Damping Test 5.D.4

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>14.8</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>14.3</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>16.5</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 12.9%
Appendix G

Damping Test 5.D.4R

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (%)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>16.2</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>17.3</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>19.1</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 11.4%
Appendix G
Damping Test 6.D.0

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 4.25 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>6.3</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>5.3</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>5.3</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 5.5%
Appendix G

Damping Test 6.D.1

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 4.25 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td></td>
<td>D19</td>
<td>14.1</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D14</td>
<td>11.3</td>
<td>D16</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D4</td>
<td>11.6</td>
<td>D6</td>
</tr>
<tr>
<td>Floor</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Mean Damping = 10.8%
Appendix G

Damping Test 6.D.2

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>5.1</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>5.6</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>5.6</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 6.4%
Appendix G

Damping Test 6.D.3

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 4.05 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>6.7</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>6.7</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>6.8</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 7.2%
Appendix G

Damping Test 6.D.4

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

### Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (%)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>7.9</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>5.6</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>5.9</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 7.0%
Appendix G

Damping Test 6.D.4R

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 3.71 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>10.5</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>10.8</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>9.5</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 10.2%
Appendix G

Damping Test 6A.D.2

Two stories with roof, east & west sheathed with window and small door openings with "wastewall" sheathing removed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 3.27 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>7.2</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>4.8</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>7.6</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 6.5%
Appendix G

Damping Test 6A.D.3

Two stories with roof, east & west sheathed with window and small door openings with "wastewall" sheathing removed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 3.22 Hz)

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall Channel</th>
<th>Damping (% Critical)</th>
<th>Center Line Channel</th>
<th>Damping (% Critical)</th>
<th>East Wall Channel</th>
<th>Damping (% Critical)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>8.4</td>
<td>D21</td>
<td>8.5</td>
<td>D23</td>
<td>6.7</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>5.7</td>
<td>D16</td>
<td>N/A</td>
<td>D18</td>
<td>5.8</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>7.7</td>
<td>D6</td>
<td>9.7</td>
<td>D8</td>
<td>6.7</td>
</tr>
</tbody>
</table>

Mean Damping = 7.4%
Appendix G

Damping Test 7.D.0

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 3.91 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel D19</td>
<td>Damping (%) Critical</td>
<td>Channel D21</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>14.8</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>6.6</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>9.1</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 8.6%
Appendix G

Damping Test 7.D.1

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 3.91 Hz)

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Channel</td>
<td>Channel</td>
</tr>
<tr>
<td></td>
<td>Damping (%) Critical</td>
<td>Damping (%) Critical</td>
<td>Damping (%) Critical</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>11.8</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>8.8</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>9.6</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 9.2%
Appendix G

Damping Test 7.D.2

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 3.91 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall Channel</th>
<th>Damping (% Critical)</th>
<th>Center Line Channel</th>
<th>Damping (% Critical)</th>
<th>East Wall Channel</th>
<th>Damping (% Critical)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>9.2</td>
<td>D21</td>
<td>10.5</td>
<td>D23</td>
<td>7.6</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>11.4</td>
<td>D16</td>
<td>10.6</td>
<td>D18</td>
<td>9.7</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>10.0</td>
<td>D6</td>
<td>9.7</td>
<td>D8</td>
<td>11.7</td>
</tr>
</tbody>
</table>

Mean Damping = 10.0%
Appendix G
Damping Test 7.D.3

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>2.7</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>9.5</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>6.5</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 7.1%
Appendix G

Damping Test 7.D.3R

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 3.56 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (%) Critical</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>10.3</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>6.9</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>9.5</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 8.3%
Appendix G

Damping Test 7.D.4

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

### Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (%)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>9.1</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>6.9</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>8.9</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 8.3%
Appendix G

Damping Test 7A.D.2

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design with anchor bolt adjacent to door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 3.37 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>Channel</td>
<td>Damping (%) Critical</td>
<td>Channel</td>
<td>Damping (%) Critical</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>13.1</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>6.3</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>7.3</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 7.7%
Appendix G

Damping Test 7A.D.3

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design with anchor bolt adjacent to door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>6.6</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>6.4</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>6.7</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 6.7%
Appendix G

Damping Test 7A.D.4

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design with anchor bolt adjacent to door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 3.37 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Channel</td>
<td>Channel</td>
</tr>
<tr>
<td></td>
<td>Damping (% Critical)</td>
<td>Damping (% Critical)</td>
<td>Damping (% Critical)</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>D21</td>
<td>D23</td>
</tr>
<tr>
<td></td>
<td>8.8</td>
<td>7.3</td>
<td>8.4</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>D16</td>
<td>D18</td>
</tr>
<tr>
<td></td>
<td>5.5</td>
<td>5.5</td>
<td>6.2</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>D6</td>
<td>D8</td>
</tr>
<tr>
<td></td>
<td>8.5</td>
<td>7.5</td>
<td>7.8</td>
</tr>
</tbody>
</table>

Mean Damping = 7.3%
Appendix G

Damping Test 8.D.0

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 4.15 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>4.8</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>4.9</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>9.3</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 6.3%
Appendix G

Damping Test 8.D.1

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Channel</td>
<td>Channel</td>
</tr>
<tr>
<td></td>
<td>Damping (%) Critical</td>
<td>Damping (%) Critical</td>
<td>Damping (%) Critical</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>D21</td>
<td>D23</td>
</tr>
<tr>
<td></td>
<td>7.2</td>
<td>7.7</td>
<td>7.2</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>D16</td>
<td>D18</td>
</tr>
<tr>
<td></td>
<td>7.8</td>
<td>6.5</td>
<td>9.0</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>D6</td>
<td>D8</td>
</tr>
<tr>
<td></td>
<td>7.4</td>
<td>8.7</td>
<td>9.9</td>
</tr>
</tbody>
</table>

Mean Damping = 7.9%
Appendix G
Damping Test 8.D.2

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 4.05 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>5.0</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>9.8</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>10.9</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 9.4%
Appendix G

Damping Test 8.D.3

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 3.91 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (%)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>8.8</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>9.9</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>8.3</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 10.5%
Appendix G

Damping Test 8.D.3R

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

### Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>8.6</td>
<td>D6</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>6.8</td>
<td>D16</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>6.7</td>
<td>D21</td>
</tr>
</tbody>
</table>

Mean Damping = 8.3%
Appendix G

Damping Test 8.D.4

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall Channel</th>
<th>Damping (% Critical)</th>
<th>Center Line Channel</th>
<th>Damping (% Critical)</th>
<th>East Wall Channel</th>
<th>Damping (% Critical)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>8.3</td>
<td>D21</td>
<td>6.1</td>
<td>D23</td>
<td>7.5</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>6.4</td>
<td>D16</td>
<td>6.4</td>
<td>D18</td>
<td>6.1</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>10.3</td>
<td>D6</td>
<td>6.3</td>
<td>D8</td>
<td>6.7</td>
</tr>
</tbody>
</table>

Mean Damping = 7.1%
Appendix G

Damping Test 9.D.0

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 3.96 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (%)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>6.2</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>4.3</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>1.8</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 4.2%
Appendix G

Damping Test 9.D.1

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 3.91 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (%) Critical</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>4.1</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>4.0</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>4.3</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 4.3%
Appendix G

Damping Test 9.D.2

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL-400 adhesive.

Damping in Fundamental Mode (Frequency = 3.71 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>5.4</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>3.7</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>3.8</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 4.2%
Appendix G

Damping Test 9.D.3

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL-400 adhesive.

Damping in Fundamental Mode (Frequency = 3.66 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (%) Critical</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>4.6</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>3.6</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>3.6</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 3.9%
Appendix G

Damping Test 9.D.3R

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 3.42 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>6.0</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>5.5</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>7.1</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 6.2%
Appendix G

Damping Test 9.D.4

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 2.93 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th></th>
<th>Center Line</th>
<th></th>
<th>East Wall</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (%)</td>
<td>Channel</td>
<td>Damping (%)</td>
<td>Channel</td>
<td>Damping (%)</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>7.0</td>
<td>D21</td>
<td>3.3</td>
<td>D23</td>
<td>3.9</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>6.9</td>
<td>D16</td>
<td>7.3</td>
<td>D18</td>
<td>7.6</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>6.9</td>
<td>D6</td>
<td>9.8</td>
<td>D8</td>
<td>12.1</td>
</tr>
</tbody>
</table>

Mean Damping = 7.2%
Appendix G

Damping Test 9.D.5

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Damping in Fundamental Mode (Frequency = 2.93 Hz)

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>10.8</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>7.2</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>5.7</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 8.7%
Appendix G

Damping Test 10.D.0

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Damping in Fundamental Mode (Frequency = 6.49 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>3.6</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>2.9</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>3.2</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 3.3%
Appendix G

Damping Test 10.D.1

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>6.2</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>6.8</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>7.6</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 6.1%
Appendix G

Damping Test 10.D.2

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Damping in Fundamental Mode (Frequency = 6.10 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall Channel</th>
<th>Damping (% Critical)</th>
<th>Center Line Channel</th>
<th>Damping (% Critical)</th>
<th>East Wall Channel</th>
<th>Damping (% Critical)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D19</td>
<td>8.3</td>
<td>D21</td>
<td>10.8</td>
<td>D23</td>
<td>13.3</td>
</tr>
<tr>
<td></td>
<td>D14</td>
<td>11.0</td>
<td>D16</td>
<td>13.1</td>
<td>D18</td>
<td>11.8</td>
</tr>
<tr>
<td></td>
<td>D4</td>
<td>11.6</td>
<td>D6</td>
<td>12.8</td>
<td>D8</td>
<td>10.9</td>
</tr>
</tbody>
</table>

Mean Damping = 11.5%
Appendix G

Damping Test 10.D.3

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (%)</td>
<td>Channel</td>
<td>Damping (%)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Critical</td>
<td></td>
<td>Critical</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>9.1</td>
<td>D21</td>
<td>8.1</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>4.5</td>
<td>D16</td>
<td>9.1</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>9.8</td>
<td>D6</td>
<td>8.7</td>
</tr>
</tbody>
</table>

Mean Damping = 8.1%
Appendix G

Damping Test 10.D.4

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>11.5</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>9.7</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>7.8</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 7.4%
Appendix G

Damping Test 10.D.4R

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Damping in Fundamental Mode (Frequency = 5.47 Hz)

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (%)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>4.5</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>5.4</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>5.2</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 5.9%
Appendix G

Damping Test 10.D.5

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Damping Summary

<table>
<thead>
<tr>
<th>Location</th>
<th>West Wall</th>
<th>Center Line</th>
<th>East Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Channel</td>
<td>Damping (% Critical)</td>
<td>Channel</td>
</tr>
<tr>
<td>Roof Ridge</td>
<td>D19</td>
<td>6.1</td>
<td>D21</td>
</tr>
<tr>
<td>Roof Eave</td>
<td>D14</td>
<td>5.4</td>
<td>D16</td>
</tr>
<tr>
<td>Floor</td>
<td>D4</td>
<td>6.1</td>
<td>D6</td>
</tr>
</tbody>
</table>

Mean Damping = 6.2%
Appendix H  Selected Seismic Results – Shake Table Fidelity
Appendix H

Shake Table Fidelity

Phase 5

Seismic Test 5.S.1

Seismic Test 5.S.2

Seismic Test 5.S.3

Seismic Test 5.S.4

Seismic Test 5.S.4R

Period (s)

Spectral Acceleration, $S_A (g)$

5% Damping

Feedback

Reference

Period (s)

Spectral Acceleration, $S_A (g)$

5% Damping

Feedback

Reference

Period (s)

Spectral Acceleration, $S_A (g)$

5% Damping

Feedback

Reference

Period (s)

Spectral Acceleration, $S_A (g)$

5% Damping

Feedback

Reference
Appendix H
Shake Table Fidelity
Phases 6 & 6A

Seismic Test 6.S.1
Seismic Test 6.S.2
Seismic Test 6.S.3
Seismic Test 6.S.4
Seismic Test 6.S.4R
Seismic Test 6A.S.3
Appendix H
Shake Table Fidelity
Phases 7 & 7A

Seismic Test 7.S.1

Seismic Test 7.S.2

Seismic Test 7.S.3

Seismic Test 7.S.3R

Seismic Test 7.S.4

Seismic Test 7A.S.3

Seismic Test 7A.S.4
Appendix H
Shake Table Fidelity

Phase 8

Seismic Test 8.S.1

Seismic Test 8.S.2

Seismic Test 8.S.3

Seismic Test 8.S.3R

Seismic Test 8.S.4

Seismic Test 7A.S.3

Seismic Test 7A.S.4
Appendix H

Shake Table Fidelity

Phase 9

Seismic Test 9.S.1

Seismic Test 9.S.2

Seismic Test 9.S.3

Seismic Test 9.S.3R

Seismic Test 9.S.4

Seismic Test 9.S.5
Appendix H
Shake Table Fidelity

Phase 10

Seismic Test 10.S.1

Seismic Test 10.S.2

Seismic Test 10.S.3

Seismic Test 10.S.4

Seismic Test 10.S.4R

Seismic Test 10.S.5

Seismic Test 10.S.5R
Appendix I  Selected Seismic Results – Visual Damage
Appendix I

Phase 5

Initial split in first-story west wall sill plate at south end

Initial split in holdown stud in first-story north wall

Split in holdown stud in first-story north wall after Level 2

Split in holdown stud in first-story east wall at south end after Level 2

Split in sill plate in first-story west wall at north end after Level 2

Split in sill plate in first-story east wall at south end after Level 2

Split in sill plate in second-story east wall at south end after Level 2

Split in sill plate in first-story north wall after Level 4

Nail holes in panel edge stud after sheathing was removed showing vertical elongation
Appendix I

Phase 6

Initial split in inter-story holdown stud due to nailing of CS16 strap

Initial split in holdown stud in second-story east wall at south end

Split in sill plate in first-story north wall after Level 2

Split in sill plate in first-story north wall after Level 2

Crushing of foundation grout at northeast corner of structure after Level 3

Split in sill plate in first-story north wall after Level 4

Split in holdown stud in second-story south wall after Level 4

Split in sill plate in first-story north wall after Level 4
Appendix I

Phases 7 & 7A

Initial splits in sill plate in second-story west wall due to nailing of sill plate

Split in stud in first-story north wall after Level 3

Split in sill plate in second-story south wall after Level 3

Split in sill plate in first-story north wall after Level 4

Cracking of sheathing at southeast corner of east door opening after Phase 7A Level 4

Cracking of sheathing at northeast corner of east door opening after Phase 7A Level 4

Cracking of sheathing at northwest corner of west door opening after Phase 7A Level 4

Cracking of sheathing at southwest corner of west door opening after Phase 7A Level 4
Appendix I

Phase 8

Text:

Head of duplex headed nail crushing sill plate at second-story east wall after Level 3

Split in blocking in east wall at floor diaphragm after Level 3

Cracking of sheathing at southeast corner of east door opening after Level 4

Cracking of sheathing at northeast corner of east door opening after Level 4

Cracking of sheathing at northwest corner of west door opening after Level 4

Cracking of sheathing at southwest corner of west door opening after Level 4

Crushing of foundation grout at north end of east door opening after Level 4

Crushing of foundation grout at north end of west door opening after Level 4
Appendix I

Phase 9

Split in blocking in east wall at floor diaphragm after Level 3

Split in sill plate in first-story north wall after Level 4

Split in holdown stud in first-story west wall at north end after Level 4

Cracking of sheathing at southeast corner of east garage opening after Level 4

Cracking of sheathing at northeast corner of east garage opening after Level 4

Cracking of sheathing at northwest corner of west door opening after Level 5

Cracking of sheathing at southwest corner of west door opening after Level 5

Buckling of sheathing panel in first-story east wall after Level 5
Appendix I

Phase 10

Hairline cracking in stucco around second-story north wall window after Level 2

Hairline cracking in stucco around second-story east wall windows after Level 2

Cracking of gypsum wallboard at interior bearing wall & GLB after Level 4

Cracking of gypsum wallboard at interior bearing wall after Level 4

Cracking of gypsum wallboard at interior bearing wall & GLB after Level 5R

Cracking of gypsum wallboard at southeast corner of east garage opening after Level 5R

Cracking of gypsum wallboard at northeast corner of east garage opening after Level 5R

Cracking and spalling of stucco at northeast corner of east garage opening after Level 5R
Appendix J  Selected Seismic Results – Displacement and Acceleration Time-Histories
Appendix J
Seismic Test 5.S.1

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 5.S.2

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 5.S.3

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 5.S.4

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 5.S.4R

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 6.S.1

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J

Seismic Test 6.S.2

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J

Seismic Test 6.S.3

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.
Appendix J
Seismic Test 6.S.4

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 6.S.4R

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J

Seismic Test 6A.S.3

Two stories with roof, east & west sheathed with window and small door openings with "wastewall" sheathing removed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

East Elevation  

West Elevation

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J

Seismic Test 7.S.1

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 7.S.2

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.
Appendix J
Seismic Test 7.S.3

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.
Appendix J

Seismic Test 7.S.3R

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

**Displacement and Acceleration Channel Locations**

**Relative Displacement Time-Histories**

**Absolute Acceleration Time-Histories**
Appendix J
Seismic Test 7.S.4

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 7A.S.3

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design with anchor bolt adjacent to door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 7A.S.4

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design with anchor bolt adjacent to door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J

Seismic Test 8.S.1

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J

Seismic Test 8.S.2

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 8.S.3

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 8.S.3R

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.
Appendix J
Seismic Test 8.S.4

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 9.S.1

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 9.S.2

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 9.S.3

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 9.S.3R

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 9.S.4

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.
Appendix J
Seismic Test 9.S.5

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 10.S.1

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J

Seismic Test 10.S.2

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 10.S.3

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J

Seismic Test 10.S.4

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J

Seismic Test 10.S.4R

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J

Seismic Test 10.S.5

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Displacement and Acceleration Channel Locations

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix J
Seismic Test 10.S.5R

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Relative Displacement Time-Histories

Absolute Acceleration Time-Histories
Appendix K  Selected Seismic Results – Base Shear Force-Displacement Hysteresis Loops
Appendix K

Phase 5

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note that this base shear is the total force induced in the foundation of the test structure including the nonlinear restoring force and the viscous damping force.
Appendix K

Phase 6

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note that this base shear is the total force induced in the foundation of the test structure including the nonlinear restoring force and the viscous damping force.
Appendix K
Phase 6A

Two stories with roof, east & west sheathed with window and small door openings with "wastewall" sheathing removed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note that this base shear is the total force induced in the foundation of the test structure including the nonlinear restoring force and the viscous damping force.
Appendix K

Phase 7

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note that this base shear is the total force induced in the foundation of the test structure including the nonlinear restoring force and the viscous damping force.
Appendix K
Phase 7A

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design with anchor bolt adjacent to door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note that this base shear is the total force induced in the foundation of the test structure including the nonlinear restoring force and the viscous damping force.
Appendix K

Phase 8

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note that this base shear is the total force induced in the foundation of the test structure including the nonlinear restoring force and the viscous damping force.
Appendix K

Phase 9

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note that this base shear is the total force induced in the foundation of the test structure including the nonlinear restoring force and the viscous damping force.
Appendix K

Phase 10

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Note that this base shear is the total force induced in the foundation of the test structure including the nonlinear restoring force and the viscous damping force.
Appendix L  Selected Seismic Results – Peak Anchor Bolt Forces
Appendix L

Seismic Test 5.S.1

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds.
Appendix L
Seismic Test 5.S.2

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

[Diagram showing peak anchor bolt forces with legend and note]

Note: All numerical values represent force in pounds.
Appendix L
Seismic Test 5.S.3

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds.
Appendix L

Seismic Test 5.S.4

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces
Appendix L
Seismic Test 5.S.4R

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds
Appendix L

Seismic Test 6.S.1

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

☑ Holdown Force
☑ Sill Anchor Bolt Force
☑ Bad Data

Note: All numerical values represent force in pounds.
Appendix L
Seismic Test 6.S.2

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

[Diagram showing peak anchor bolt forces with labels and notes]

Note: All numerical values represent force in pounds.
Appendix L

Seismic Test 6.S.3

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds
Appendix L

Seismic Test 6.S.4

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

☐ Holdown Force
☒ Sill Anchor Bolt Force
■ Bad Data

Note: All numerical values represent force in pounds
Appendix L

Seismic Test 6.S.4R

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds
Appendix L

Seismic Test 6A.S.3

Two stories with roof, east & west sheathed with window and small door openings with "wastewall" sheathing removed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds
Appendix L
Seismic Test 7.S.1

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces
Appendix L

Seismic Test 7.S.2

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

[Diagram showing anchor bolt forces with North orientation and various values indicated.]
Appendix L

Seismic Test 7.S.3

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds.
Appendix L
Seismic Test 7.S.3R

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

- Holdown Force
- Sill Anchor Bolt Force
- Bad Data

Note: All numerical values represent force in pounds
Appendix L

Seismic Test 7.S.4

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds.
Appendix L

Seismic Test 7A.S.3

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design with anchor bolt adjacent to door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

[Diagram showing peak anchor bolt forces with labels and values.]

Note: All numerical values represent force in pounds.
Appendix L

Seismic Test 7A.S.4

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design with anchor bolt adjacent to door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds
Appendix L
Seismic Test 8.S.1

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds
Appendix L

Seismic Test 8.S.2

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds.
Appendix L
Seismic Test 8.S.3

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds.
Appendix L

Seismic Test 8.S.3R

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds.
Appendix L
Seismic Test 8.S.4

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces
Appendix L

Seismic Test 9.S.1

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

[Diagram showing anchor bolt forces with symbols and numbers, including a note that all numerical values represent force in pounds.]
Appendix L

Seismic Test 9.S.2

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

- Holdown Force
- Sill Anchor Bolt Force
- Bad Data

Note: All numerical values represent force in pounds
Appendix L
Seismic Test 9.S.3

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds.
Appendix L

Seismic Test 9.S.3R

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

□ Holdown Force
☑ Sill Anchor Bolt Force
■ Bad Data

Note: All numerical values represent force in pounds
Appendix L

Seismic Test 9.S.4

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces
Appendix L
Seismic Test 9.S.5

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds

- Holdown Force
- Stiff Anchor Bolt Force
- Bad Data

North
Appendix L

Seismic Test 10.S.1

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Anchor Bolt Forces

- Holdown Force
- Sill Anchor Bolt Force
- Bad Data

Note: All numerical values represent force in pounds
Appendix L
Seismic Test 10.S.2

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Anchor Bolt Forces

[Diagram showing anchor bolt forces with labels and notes for interpretation]

Legend:
- Holdown Force
- Sill Anchor Bolt Force
- Bad Data

Note: All numerical values represent force in pounds
Appendix L
Seismic Test 10.S.3

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds.

Legend:
- Holdown Force
- Sill Anchor Bolt Force
- Bad Data
Appendix L
Seismic Test 10.S.4
Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds
Appendix L

Seismic Test 10.S.4R

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds
Appendix L
Seismic Test 10.S.5

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Anchor Bolt Forces

Note: All numerical values represent force in pounds.
Appendix L
Seismic Test 10.S.5R

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Anchor Bolt Forces

North
Appendix M  Selected Seismic Results – Peak
Sill Plate and Holdown Stud Uplift
Appendix M
Seismic Test 5.S.1

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M
Seismic Test 5.S.2

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M
Seismic Test 5.S.3

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M
Seismic Test 5.S.4

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M
Seismic Test 5.S.4R

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M
Seismic Test 6.S.1

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 6.S.2

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 6.S.3

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 6.S.4

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M
Seismic Test 6.S.4R

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M
Seismic Test 6A.S.3

Two stories with roof, east & west sheathed with window and small door openings with "wastewall" sheathing removed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 7.S.1

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 7.S.2

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate and Holdown Stud Uplift

2nd Story Uplift Configuration

1st Story Uplift Configuration

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 7.S.3

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M
Seismic Test 7.S.3R

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 7.S.4

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M
Seismic Test 7A.S.3

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design with anchor bolt adjacent to door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate and Holdown Stud Uplift

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M
Seismic Test 7A.S.4

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design with anchor bolt adjacent to door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 8.S.1

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 8.S.2

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 8.S.3

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate and Holdown Stud Uplift

2nd Story Uplift Configuration

1st Story Uplift Configuration

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 8.S.3R

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate and Holdown Stud Uplift

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M
Seismic Test 8.S.4

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate and Holdown Stud Uplift

2nd Story Uplift Configuration

1st Story Uplift Configuration

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 9.S.1

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate and Holdown Stud Uplift

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 9.S.2

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate and Holdown Stud Uplift

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M
Seismic Test 9.S.3

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate and Holdown Stud Uplift

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 9.S.3R

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

**Note:** The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 9.S.4

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate and Holdown Stud Uplift

2nd Story Uplift Configuration

First Story Uplift Configuration

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 9.S.5

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate and Holdown Stud Uplift

2nd Story Uplift Configuration

1st Story Uplift Configuration

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M
Seismic Test 10.S.1

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Sill Plate and Holdown Stud Uplift

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 10.S.2

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Sill Plate and Holdown Stud Uplift

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 10.S.3

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Sill Plate and Holdown Stud Uplift

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 10.S.4

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Sill Plate and Holdown Stud Uplift

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 10.S.4R

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

**Peak Sill Plate and Holdown Stud Uplift**

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 10.S.5

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Sill Plate and Holdown Stud Uplift

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix M

Seismic Test 10.S.5R

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Sill Plate and Holdown Stud Uplift

Note: The bottom numerical value represents the peak sill plate uplift and the top numerical value represents the total uplift (sill plate + holdown stud). Only the peak sill plate uplift value is shown if the holdown stud uplift was zero. All values are in inches.
Appendix N  Selected Seismic Results – Sill Plate Slippage
Appendix N
Seismic Test 5.S.1

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  - Positive slippage is in the north direction for the east and west walls
  - Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  - Negative slippage is in the south direction for the east and west walls
  - Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 5.S.2

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 5.S.3

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 5.S.4

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N
Seismic Test 5.S.4R

Two stories with roof, east & west fully sheathed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 6.S.1

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 6.S.2

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 6.S.3

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  - Positive slippage is in the north direction for the east and west walls
  - Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  - Negative slippage is in the south direction for the east and west walls
  - Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 6.S.4

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  - Positive slippage is in the north direction for the east and west walls
  - Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  - Negative slippage is in the south direction for the east and west walls
  - Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N
Seismic Test 6.S.4R

Two stories with roof, east & west sheathed with window and small door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

☐ Peak Positive Sill Plate Slippage
Positive slippage is in the north direction for the east and west walls
Positive slippage is in the east direction for the north and south walls

☐ Peak Negative Sill Plate Slippage
Negative slippage is in the south direction for the east and west walls
Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N  
Seismic Test 6A.S.3  
Two stories with roof, east & west sheathed with window and small door openings with "wastewall" sheathing removed, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 7.S.1

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N
Seismic Test 7.S.2
Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 7.S.3

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  - Positive slippage is in the north direction for the east and west walls
  - Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  - Negative slippage is in the south direction for the east and west walls
  - Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N
Seismic Test 7.S.3R

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls
Appendix N
Seismic Test 7.S.4

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Diagram:

- **Peak Positive Sill Plate Slippage**
  - Positive slippage is in the north direction for the east and west walls
  - Positive slippage is in the east direction for the north and south walls

- **Peak Negative Sill Plate Slippage**
  - Negative slippage is in the south direction for the east and west walls
  - Negative slippage is in the west direction for the north and south walls

*Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.*
Appendix N

Seismic Test 7A.S.3

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design with anchor bolt adjacent to door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N
Seismic Test 7A.S.4

Two stories with roof, east & west sheathed with window and small door openings using perforated shearwall design with anchor bolt adjacent to door openings, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 8.S.1

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 8.S.2

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N
Seismic Test 8.S.3

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 8.S.3R

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls
Appendix N
Seismic Test 8.S.4

Two stories with roof, east & west sheathed with window and small door openings using conventional construction, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls
Appendix N
Seismic Test 9.S.1

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 9.S.2

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  - Positive slippage is in the north direction for the east and west walls
  - Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  - Negative slippage is in the south direction for the east and west walls
  - Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 9.S.3

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N
Seismic Test 9.S.3R

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N
Seismic Test 9.S.4

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N
Seismic Test 9.S.5

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N
Seismic Test 10.S.1

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  - Positive slippage is in the north direction for the east and west walls
  - Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  - Negative slippage is in the south direction for the east and west walls
  - Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 10.S.2

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall. North & south sheathed with openings. Second floor diaphragm nailed at 100% with no blocking & PL-400 adhesive. Finished with exterior stucco and interior gypsum wallboard.

**Peak Sill Plate Slippage**

- **Peak Positive Sill Plate Slippage**
  - Positive slippage is in the north direction for the east and west walls.
  - Positive slippage is in the east direction for the north and south walls.

- **Peak Negative Sill Plate Slippage**
  - Negative slippage is in the south direction for the east and west walls.
  - Negative slippage is in the west direction for the north and south walls.

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 10.S.3

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N
Seismic Test 10.S.4

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 10.S.4R

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 10.S.5

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  - Positive slippage is in the north direction for the east and west walls
  - Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  - Negative slippage is in the south direction for the east and west walls
  - Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix N

Seismic Test 10.S.5R

Two stories with roof, east & west sheathed with window openings and small door opening on west wall and large door opening on east wall, north & south sheathed with openings, second floor diaphragm nailed at 100% with no blocking & PL400 adhesive, finished with exterior stucco and interior gypsum wallboard.

Peak Sill Plate Slippage

- Peak Positive Sill Plate Slippage
  Positive slippage is in the north direction for the east and west walls
  Positive slippage is in the east direction for the north and south walls

- Peak Negative Sill Plate Slippage
  Negative slippage is in the south direction for the east and west walls
  Negative slippage is in the west direction for the north and south walls

Note: Sill plate slippage measured relative to base on 1st story and relative to floor sheathing on 2nd story. All values are in inches.
Appendix O  Comparison of Experimental and Numerical Results for Phase 9
Appendix O
Phase 9 - Level 1
Displacement Time-Histories

1st Story East Wall
Experimental

1st Story West Wall
Experimental

2nd Story East Wall
Experimental

2nd Story West Wall
Experimental

Numerical
Appendix O
Phase 9 - Level 1
Acceleration Time-Histories
Appendix O

Phase 9 - Level 2
Displacement Time-Histories

1st Story East Wall

Experimental

Relative Displacement (in)

0.34 in

-0.24 in

Numerical

0.53 in

-0.41 in

Time (s)

1.00

0.50

0.00

-0.50

-1.00

0 5 10 15 20 25

1st Story West Wall

Experimental

0.28 in

-0.16 in

Numerical

0.37 in

-0.28 in

Time (s)

1.00

0.50

0.00

-0.50

-1.00

0 5 10 15 20 25

2nd Story East Wall

Experimental

0.60 in

-0.39 in

Numerical

0.82 in

-0.65 in

Time (s)

1.00

0.50

0.00

-0.50

-1.00

0 5 10 15 20 25

2nd Story West Wall

Experimental

0.49 in

-0.33 in

Numerical

0.65 in

-0.51 in

Time (s)

1.00

0.50

0.00

-0.50

-1.00

0 5 10 15 20 25
Appendix O
Phase 9 - Level 2
Acceleration Time-Histories

1st Story East Wall

<table>
<thead>
<tr>
<th>Time (s)</th>
<th>Experimental</th>
<th>Numerical</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.35 g</td>
<td>0.41 g</td>
</tr>
<tr>
<td>5</td>
<td>-0.25 g</td>
<td>-0.31 g</td>
</tr>
<tr>
<td>10</td>
<td>0.00 g</td>
<td>0.00 g</td>
</tr>
<tr>
<td>15</td>
<td>0.40 g</td>
<td>0.31 g</td>
</tr>
<tr>
<td>20</td>
<td>0.80 g</td>
<td>0.33 g</td>
</tr>
<tr>
<td>25</td>
<td>0.00 g</td>
<td>0.00 g</td>
</tr>
</tbody>
</table>

1st Story West Wall

<table>
<thead>
<tr>
<th>Time (s)</th>
<th>Experimental</th>
<th>Numerical</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.31 g</td>
<td>0.38 g</td>
</tr>
<tr>
<td>5</td>
<td>-0.21 g</td>
<td>-0.31 g</td>
</tr>
<tr>
<td>10</td>
<td>0.00 g</td>
<td>0.00 g</td>
</tr>
<tr>
<td>15</td>
<td>0.40 g</td>
<td>0.38 g</td>
</tr>
<tr>
<td>20</td>
<td>0.80 g</td>
<td>0.48 g</td>
</tr>
<tr>
<td>25</td>
<td>0.00 g</td>
<td>0.00 g</td>
</tr>
</tbody>
</table>

2nd Story East Wall

<table>
<thead>
<tr>
<th>Time (s)</th>
<th>Experimental</th>
<th>Numerical</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.58 g</td>
<td>0.41 g</td>
</tr>
<tr>
<td>5</td>
<td>-0.49 g</td>
<td>-0.39 g</td>
</tr>
<tr>
<td>10</td>
<td>0.00 g</td>
<td>0.00 g</td>
</tr>
<tr>
<td>15</td>
<td>0.40 g</td>
<td>0.39 g</td>
</tr>
<tr>
<td>20</td>
<td>0.80 g</td>
<td>0.40 g</td>
</tr>
<tr>
<td>25</td>
<td>0.00 g</td>
<td>0.00 g</td>
</tr>
</tbody>
</table>

2nd Story West Wall

<table>
<thead>
<tr>
<th>Time (s)</th>
<th>Experimental</th>
<th>Numerical</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.48 g</td>
<td>0.41 g</td>
</tr>
<tr>
<td>5</td>
<td>-0.38 g</td>
<td>-0.39 g</td>
</tr>
<tr>
<td>10</td>
<td>0.00 g</td>
<td>0.00 g</td>
</tr>
<tr>
<td>15</td>
<td>0.40 g</td>
<td>0.40 g</td>
</tr>
<tr>
<td>20</td>
<td>0.80 g</td>
<td>0.40 g</td>
</tr>
<tr>
<td>25</td>
<td>0.00 g</td>
<td>0.00 g</td>
</tr>
</tbody>
</table>
Appendix O
Phase 9 - Level 3
Displacement Time-Histories

1st Story East Wall
Experimental

Relative Displacement (in)

0.77 in
-0.60 in

Numerical

1.03 in
-0.54 in

Relative Displacement (in)

Time (s)

0 5 10 15 20 25

1st Story West Wall
Experimental

Relative Displacement (in)

0.58 in
-0.45 in

Numerical

0.74 in
-0.37 in

Relative Displacement (in)

Time (s)

0 5 10 15 20 25

2nd Story East Wall
Experimental

Relative Displacement (in)

1.31 in
-1.05 in

Numerical

1.37 in
-0.79 in

Relative Displacement (in)

Time (s)

0 5 10 15 20 25

2nd Story West Wall
Experimental

Relative Displacement (in)

1.10 in
-0.89 in

Numerical

1.06 in
-0.63 in

Relative Displacement (in)

Time (s)

0 5 10 15 20 25
Appendix O
Phase 9 - Level 3
Acceleration Time-Histories

1st Story East Wall
Experimental

Absolute Acceleration (g)

Time (s)

0.52 g
-0.37 g
0.00

1st Story West Wall
Experimental

Absolute Acceleration (g)

Time (s)

0.46 g
-0.34 g
0.00

1st Story East Wall
Numerical

Absolute Acceleration (g)

Time (s)

0.62 g
-0.53 g
0.00

1st Story West Wall
Numerical

Absolute Acceleration (g)

Time (s)

0.51 g
-0.49 g
0.00

2nd Story East Wall
Experimental

Absolute Acceleration (g)

Time (s)

0.73 g
-0.70 g
0.00

2nd Story West Wall
Experimental

Absolute Acceleration (g)

Time (s)

0.70 g
-0.58 g
0.00

2nd Story East Wall
Numerical

Absolute Acceleration (g)

Time (s)

0.83 g
-0.57 g
0.00

2nd Story West Wall
Numerical

Absolute Acceleration (g)

Time (s)

0.70 g
-0.49 g
0.00
Appendix O

Phase 9 - Level 3R

Displacement Time-Histories

<table>
<thead>
<tr>
<th></th>
<th>1st Story East Wall</th>
<th>1st Story West Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Exper.</strong></td>
<td>0.94 in</td>
<td>0.74 in</td>
</tr>
<tr>
<td><strong>Numerical</strong></td>
<td>-0.60 in</td>
<td>-0.47 in</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>2nd Story East Wall</th>
<th>2nd Story West Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Exper.</strong></td>
<td>1.28 in</td>
<td>1.39 in</td>
</tr>
<tr>
<td><strong>Numerical</strong></td>
<td>-0.57 in</td>
<td>-0.91 in</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>1st Story East Wall</th>
<th>1st Story West Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Exper.</strong></td>
<td>1.61 in</td>
<td>1.30 in</td>
</tr>
<tr>
<td><strong>Numerical</strong></td>
<td>-1.05 in</td>
<td>-0.59 in</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>2nd Story East Wall</th>
<th>2nd Story West Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Exper.</strong></td>
<td>1.62 in</td>
<td>1.30 in</td>
</tr>
<tr>
<td><strong>Numerical</strong></td>
<td>-0.76 in</td>
<td>-0.59 in</td>
</tr>
</tbody>
</table>
Phase 9 - Level 3R
Acceleration Time-Histories

1st Story East Wall

Experimental

Numerical

1st Story West Wall

Experimental

Numerical

2nd Story East Wall

Experimental

Numerical

2nd Story West Wall

Experimental

Numerical
Appendix O
Phase 9 - Level 4
Displacement Time-Histories

1st Story East Wall
- Experimental
- Numerical

1st Story West Wall
- Experimental
- Numerical

2nd Story East Wall
- Experimental
- Numerical

2nd Story West Wall
- Experimental
- Numerical
Appendix O
Phase 9 - Level 4
Acceleration Time-Histories

1st Story East Wall
Experimental

Numerical

1st Story West Wall
Experimental

Numerical

2nd Story East Wall
Experimental

Numerical

2nd Story West Wall
Experimental

Numerical
Appendix O
Phase 9 - Level 5
Displacement Time-Histories

1st Story East Wall
Experimental

1st Story West Wall
Experimental

2nd Story West Wall
Numerical

2nd Story East Wall
Numerical
Appendix O
Phase 9 - Level 5
Acceleration Time-Histories

1st Story East Wall
 Experimental

1st Story West Wall
 Experimental

2nd Story East Wall
 Numerical

2nd Story West Wall
 Numerical

<table>
<thead>
<tr>
<th>Time (s)</th>
<th>0.65 g</th>
<th>-1.00 g</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time (s)</td>
<td>0.64 g</td>
<td>-1.11 g</td>
</tr>
<tr>
<td>Time (s)</td>
<td>1.16 g</td>
<td>-1.35 g</td>
</tr>
<tr>
<td>Time (s)</td>
<td>0.76 g</td>
<td>-1.09 g</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Time (s)</th>
<th>0.65 g</th>
<th>-0.91 g</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time (s)</td>
<td>0.63 g</td>
<td>-1.14 g</td>
</tr>
<tr>
<td>Time (s)</td>
<td>1.07 g</td>
<td>-1.27 g</td>
</tr>
<tr>
<td>Time (s)</td>
<td>0.73 g</td>
<td>-1.02 g</td>
</tr>
</tbody>
</table>
Chapter 14 References


