Development of a Testing Protocol for Woodframe Structures

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Preface

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

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

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

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

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

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

### Woodframe Project Testing and Analysis Investigations

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

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

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

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

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

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


Contents

Summary ix
Objective and Scope 1

Proposed Testing Protocols

1. Testing Protocol for Deformation Controlled Quasi-Static Cyclic Testing 2
   1.1 Loading History for Ordinary Ground Motions (Basic Loading History) 2
   1.2 Loading History for Near-Fault Ground Motions (Near-Fault Loading History) 5
   1.3 Specimen Fabrication, Testing and Instrumentation Issues 6
   1.4 Documentation of Specimens and Test Results 7

2. Testing Protocol for Force Controlled Quasi-Static Cyclic Testing 9
   2.1 Loading History for Force Controlled Quasi-Static Cyclic Testing 9
   2.2 Specimen Fabrication, Testing, Instrumentation, and Documentation 10

3. Time Histories for Shaking Table Testing 10

Figures 12

Commentary to Proposed Testing Protocols 19

C1. Considerations in Development of Loading Protocols 19
   C1.1 Considerations for Deformation-Controlled Quasi-Static Cyclic Testing 19
   C1.2 Considerations for Force Controlled Quasi-Static Cyclic Testing 20
   C1.3 Considerations for Shaking Table Testing 21

C.2 Selection of Ground Motion Records 21
   C.2.1 Set of Ordinary Records for Development of Basic Loading History 23
   C.2.2 Small Events Preceding a Performance Assessment Event 24
   C.2.3 Set of Near-Fault Records for Development of Near-Fault Loading History 25

C.3 Selection of Structural Systems for Prediction of Response 26
   C.3.1 Common System Parameters 27
   C.3.2 Hysteresis Models 28

C.4 Maximum Response Values 29
   C.4.1 Results for Ordinary Ground Motions 29
   C.4.2 Results for Near-Fault Ground Motions 31

C.5 Cumulative Damage Considerations 32
   C.5.1 Cumulative Damage Issues 32
   C.5.2 Process for Incorporating Cumulative Damage Effects into Loading History 34

C.6 Development of Representative Loading Histories 35
   C.6.1 Development of Deformation Controlled Basic Loading History 36
   C.6.2 Development of Deformation Controlled Near-Fault Loading History 38
   C.6.3 Development of Force Controlled Loading History 39

C.7 Representative Input for Shaking Table Studies 40

Acknowledgements
Summary

This report offers recommendations for a protocol for quasi-static experimentation on components of woodframe structures and for shaking table experimentation on wooden houses. The emphasis is on the development of loading histories useful for a performance assessment at various performance levels, for the evaluation of various failure modes, and for the development of design equations and analytical models. Recommendations are made also on specimen fabrication, testing procedures, specimen instrumentation, and documentation of test results.

The development of loading histories is based on results of nonlinear dynamic analysis of representative hysteretic systems subjected to sets of ordinary and near-fault ground motions. Cumulative damage concepts are employed to transform time history responses into representative deformation and force controlled loading histories. The results of the work are

- a testing protocol for deformation controlled quasi-static cyclic testing, consisting of proposed loading histories for ordinary and near-fault ground motions, and recommendations for fabrication and instrumentation of test specimens and documentation of test data,

- a testing protocol for force controlled quasi-static cyclic testing, and

- a series of recommendations on input ground motions for shaking table studies.
Objective and Scope

Several testing protocols have been proposed, and are in use, for the monotonic and cyclic testing of woodframe structural components (ASTM 1995a, ASTM 1995b, CEN 1995, CoLA/UCI Committee 1999, Foliente et al. 1998, ISO 1999, SAA 1997, Shepherd 1996). The objective of the work summarized in this report is to establish common testing protocols for all component tests and shake table tests of the CUREE/Caltech Woodframe Project. The loading histories in these testing protocols should represent the seismic demands imposed by Californian earthquakes on woodframe buildings. For shake table testing, single- and multi-axis excitations for different seismic hazard levels should be established.

The loading histories should be representative of the seismic demands imposed on components and structures for the following conditions:

- Ordinary ground motions that represent design events envisioned by present codes.
- Near-fault ground motions.
- Multiple earthquakes occurring in the lifetime of the structure.

The development of loading histories for seismic performance testing requires the execution of time history analysis that captures the demand characteristics peculiar to the wood structures of interest in the Woodframe Project. These demands are evaluated through simulation studies in which analytical models of representative woodframe structures are subjected to ground motions of various characteristics. The demands are then represented in loading histories that simulate, in a cumulative manner, the damage experienced by the structural systems. To perform this work, models of structures of different periods are subjected to various sets of ground motions, utilizing a versatile force-deformation model capable of representing the hysteretic characteristics of typical wood components and systems.

Specifically, the following tasks are addressed:

- Develop a basic loading history for component tests, considering ordinary ground motions representative of California (in particular Los Angeles) conditions.
- Develop a near-fault loading history for component tests.
- Develop a loading history for force-controlled elements (e.g., certain types of hold-downs).
- Develop a testing protocol that addresses common issues of testing and documentation of component test results.
- Establish sequential ground motions for shaking table tests.
- Establish near-fault ground motions for shaking table tests.
Proposed Testing Protocols

Testing protocols are concerned with the construction and instrumentation of test specimens, the planning and execution of experiments, the loading history to be applied to a test specimen, and the documentation of experimental results. In this report the emphasis is on the development and documentation of loading histories for deformation and force controlled component testing and of time history inputs for shaking table testing. All other aspects of testing protocols are summarized as needed and, whenever feasible, adopted by reference to existing documents.

1. Testing Protocol for Deformation Controlled Quasi-Static Cyclic Testing

This protocol is intended to apply for all component tests in which a deformation parameter (displacement, rotation, angle of shear distortion, etc.) can be identified that relates the component response to the response of the structural system of which the component is part. A typical example is the test of a plywood shear wall panel in which the racking distortion can be related to the story drift. An example to which this protocol should not be applied is the test of anchor bolts in which the interest is not in the load-deflection behavior as much as in the sudden brittle failure that would limit the system capacity.

1.1 Loading History for Ordinary Ground Motions (Basic Loading History)

The primary objective of this loading history is to evaluate capacity level seismic performance of components subjected to ordinary (not near-fault) ground motions whose probability of exceedance in 50 years is 10 percent. Deformation cycles due to smaller events prior to the capacity level event are included in the history’s deformation history. Applicability of the loading history to limit states other than capacity is not specifically addressed. The commentary provides further discussion of the objectives.

The loading history for a basic cyclic load test should follow the pattern given in Fig. 1. The history is defined by variations in deformation amplitudes, using the reference deformation $\Delta$ as the absolute measure of deformation amplitude. The history consists of

- initiation cycles,
- primary cycles, and
- trailing cycles.

Initiation cycles are executed at the beginning of the loading history. They serve to check loading equipment, measurement devices, and the force-deformation response at small amplitudes. A primary cycle is a cycle that is larger than all of the preceding cycles and is followed by smaller cycles, which are called trailing cycles. All trailing cycles have an amplitude that is equal to 75% of the amplitude of the preceding primary cycle. All cycles are symmetric, i.e., they have identical positive and negative amplitudes. Deformation control should be used throughout the experiment.

The following sequence of cycles is to be executed:
• Six cycles with an amplitude of 0.05\( \Delta \) (initiation cycles)
• A primary cycle with an amplitude of 0.075\( \Delta \)
• Six trailing cycles
• A primary cycle with an amplitude of 0.1\( \Delta \)
• Six trailing cycles
• A primary cycle with an amplitude of 0.2\( \Delta \)
• Three trailing cycles
• A primary cycle with an amplitude of 0.3\( \Delta \)
• Three trailing cycles
• A primary cycle with an amplitude of 0.4\( \Delta \)
• Two trailing cycles
• A primary cycle with an amplitude of 0.7\( \Delta \)
• Two trailing cycles
• A primary cycle with an amplitude of 1.0\( \Delta \)
• Two trailing cycles
• Increasing steps of the same pattern with an increase in amplitude of 0.5\( \Delta \), i.e., one primary cycle of amplitude equal to that of the previous primary cycle plus 0.5\( \Delta \), followed by two trailing cycles.

Reference deformation \( \Delta \):

This deformation is the maximum deformation (displacement, drift angle, rotation, etc.) the test specimen is expected to sustain according to a prescribed acceptance criterion, and assuming that the proposed basic loading history has been applied to the test specimen. It is a measure of the deformation capacity of the specimen. Thus it will be necessary to estimate the deformation capacity prior to the test. This estimate can be based on previous experience, the execution of a monotonic (or near-fault) test to assist in this estimate, or a consensus value that may prove to be useful for comparing tests of different details or configurations.

The choice of the reference value \( \Delta \) may vary from component to component or may be fixed for a specific testing program. The Woodframe Project management will provide input to this decision. The general guidelines are as follows:

- Perform a monotonic test, which provides data on the monotonic deformation capacity, \( \Delta_m \). This capacity is defined as the deformation at which the applied load drops, for the first time, below 80% of the maximum load that was applied to the specimen, see Fig. 2.
- Use a specific fraction of \( \Delta_m \), i.e. \( \gamma \Delta_m \), as the reference deformation for the basic cyclic load test. At this time, a value of \( \Delta = 0.6 \Delta_m \) is suggested. The factor \( \gamma \) should account for the difference in deformation capacity between a monotonic test and a cyclic test in which cumulative damage will lead to earlier deterioration in strength. This factor is subject to change based on information acquired in the Woodframe project testing programs.

Additional considerations:

1. The test should be continued in the predetermined pattern until the maximum load applied in a cycle decreases to a small fraction of the maximum load.
2. If deemed useful, the execution of small amplitude cycles after the execution of each “step” (a primary cycle followed by trailing cycles) should be contemplated.

3. A final definition of acceptable performance is not provided here. Testing and analysis will tell what acceptability criteria should be used. The emphasis for acceptance should be on a threshold for unacceptable deterioration in strength. A basic concept is to define the deformation level associated with acceptable performance as that primary cycle amplitude at which both of the following criteria are fulfilled for the last time:

(a) At both the positive and negative peak deformation of the primary cycle (points A and B in Fig. 1) the load does not drop below a predetermined fraction $\alpha$ of the maximum load that was applied to the specimen in the respective direction. For reporting of CUREE/Caltech Woodframe test results a value of $\alpha = 0.8$ is recommended.

(b) After the trailing cycles have been completed and the next larger primary cycle is attempted, the maximum load at deformations $\geq$ the deformation amplitude of the previous primary cycle (point C in Fig. 1) should not be less than a predetermined fraction $\beta$ of the maximum load that was applied to the specimen in the positive direction. For reporting of CUREE/Caltech Woodframe test results a value of $\beta = 0.4$ is recommended.

4. The deformation level associated with acceptable performance does not have to be equal to the selected value of $\Delta$. It may be associated with a smaller or larger deformation amplitude.

5. Acceptable performance for a variety of performance objectives other than collapse will be addressed later in the Woodframe project. Consideration might include strength, damage control, and cost of repair. Test performed in accordance with this protocol should prove useful for these performance objectives.

Potential Variations to Basic Loading History

The basic loading history, like the other loading histories described next, has been developed with an emphasis on performance evaluation. Thus, emphasis is placed on a conservative but realistic simulation of cycles that contribute significantly to damage at the 10/50 hazard level, as well as on adequate simulation of potentially damaging cycles at hazard levels associated with higher performance levels (e.g., continuous operation under more frequent events). The former necessitates the distinction between primary and trailing cycles, and the latter necessitates the execution of a large number of relatively small cycles. Both considerations render the basic loading history more complicated.

The following two options are presented as potential simplified alternatives to the basic loading history:

**Abbreviated Basic Loading History.** This loading history, which is illustrated in Fig. 3, has fewer smaller cycles. Compared to the basic loading history, the following simplifications are incorporated:

- There are four cycles with an amplitude of $0.05\Delta$ (rather than six cycles)
- There are four (rather than six) trailing cycles following the primary cycle of amplitude $0.075\Delta$
• There are four (rather than six) trailing cycles following the primary cycle of amplitude
  $0.1\Delta$
• There are two (rather than three) trailing cycles following the primary cycle of amplitude
  $0.2\Delta$
• There are two (rather than three) trailing cycles following the primary cycle of amplitude
  $0.3\Delta$

The expectation is that the smaller number of small cycles will not have a large influence on performance. Calibration testing will be needed to tell the extent of the influence.

_Simplified Basic Loading History._ This loading history is illustrated in Fig. 4. In this history, the trailing cycles, which have an amplitude of 75% of the preceding primary cycle, are replaced by cycles of an amplitude equal to that of the preceding primary cycle. Thus, several cycles of equal amplitude are being executed at each step. This simplification facilitates the execution of the test and the test interpretation, and may be more useful for the development of analytical models. But it must be recognized that it will overestimate the extent of damage, particularly for large amplitude cycles. The extent to which the damage is overestimated is not known. Only testing will tell.

**Permissible Deviation for Acceptance Testing:**

The basic loading history is a realistic and conservative representation of the cyclic deformation history to which a component of a wood structure likely is subjected in earthquakes. At relatively large deformations (primary cycles exceeding an amplitude of $0.4\Delta$), the amplitude of the primary cycles increases by steps $\geq 0.3\Delta$. These large steps are based on statistics of response deformation demands. In a testing program in which the results of different tests have to be compared and evaluated for analytical modeling, the proposed loading history should be followed rigorously and without deviations. However, if the purpose of the experiment is acceptance testing, then it is permissible to reduce the step size of the primary cycles with amplitude $> 0.4\Delta$ at the discretion of the experimentalist. But even with smaller step sizes, every primary cycle must be followed by two trailing cycles of amplitude equal to 0.75 of the preceding primary cycle. Smaller step sizes close to failure (according to an established acceptance criterion) may result in a larger capacity (largest amplitude at which the acceptance criteria are passed), even though they will result in larger cumulative damage. The reason is that the large step sizes of the basic loading history permit evaluation of acceptance only at large amplitude intervals. This is illustrated in the test example presented in Fig. 9, in which the acceptance test is passed at the target amplitude $\Delta = 0.6\Delta_m$, but is not passed at the amplitude $1.5\times0.6\Delta_m = 0.9\Delta_m$, because of the very large deterioration before the primary cycle at this amplitude is completed. But it is quite conceivable that the acceptance test would have been passed at an amplitude between 0.6 and $0.9\Delta_m$.

1.2 _Loading History for Near-Fault Ground Motions (Near-Fault Loading History)_

The loading history for the near-fault cyclic load test should follow the history given in Fig. 5. The history is defined by variations in deformation amplitudes, using the reference deformation $\Delta_n$ as the absolute measure of deformation amplitude. The history consists of the following sequence of cycles:
• Four cycles with an amplitude of 0.025$\Delta_n$
• Four cycles with an amplitude of 0.05$\Delta_n$
• A primary cycle with an amplitude of 0.1$\Delta_n$
• Two trailing cycles of amplitude 0.075$\Delta_n$
• A primary cycle with an amplitude of 0.6$\Delta_n$
• One trailing cycle with an amplitude of 0.2$\Delta_n$
• A primary positive excursion to 1.0$\Delta_n$
• A reversal to zero deformation
• A positive excursion to 0.8$\Delta_n$
• Two cycles with amplitude 0.1$\Delta_n$ and mean deformation of 0.7$\Delta_n$
• One positive excursion to the maximum deformation the test specimen can sustain without causing harm to the test facility.

Reference deformation $\Delta_0$:

This deformation is the maximum deformation (displacement, drift angle, rotation, etc.) the test specimen is expected to sustain according to a prescribed acceptance criterion, and assuming that the proposed near-fault loading history has been applied to the test specimen. It is a measure of the deformation capacity of the specimen. It will be necessary to estimate this deformation capacity prior to the test.

The present recommendation is to take this deformation capacity from a monotonic test, i.e., setting it equal to the monotonic deformation capacity $\Delta_m$ as defined in Section 1.1. It is hypothesized that the deformation capacity under the near-fault loading history will be close to that under monotonic loading. Pilot tests will be performed to test this hypothesis. Should this hypothesis prove to be reasonable, the much simpler monotonic test can be used in lieu of the more complex near-fault test. Thus, at this time the near-fault loading history is of secondary importance except for correlation tests with monotonic loading.

Additional considerations:

1. Loading should be continued monotonically after point C until the maximum load applied decreases to a small fraction of the maximum load.

2. A final definition of acceptable performance is not provided here. Testing and analysis will tell what acceptability criteria should be used. The emphasis for acceptance should be on a threshold for unacceptable deterioration in strength. A basic concept is to tie acceptability to the maximum deformation associated with a residual strength of a predetermined fraction $\gamma$ of the maximum load that was applied to the specimen in the direction of the pulse (Point A in Fig. 5). For reporting of CUREE/Caltech Woodframe test results a value of $\gamma = 0.8$ is recommended.

1.3 Specimen Fabrication, Testing and Instrumentation Issues

Recommendations for the fabrication of test specimens, for material testing, planning and execution of experiments, test control, and specimen instrumentation should be taken from
existing standards and guidelines for testing of components and materials applicable to the woodframe project. Materials of interest are wood (for framing and panel elements), stucco and gypsum (for panel elements), and steel (light gage metal studs, hold-downs, and nails). Summarized here are a few general considerations, but they should not be considered as comprehensive.

**Specimen fabrication.** Test specimens should replicate in-situ conditions so that material properties, standard construction techniques, and boundary conditions are properly simulated. Specimens should be as close as possible to full size in order to minimize size effects.

**Material Testing.** Salient properties of materials used as part of the test specimens should be measured and documented, so that the sources of damage and failure modes can be evaluated and quantified. For wood the species and grade should be recorded and the moisture content should be determined (ASTM (1995a) Section 15.5).

**Test Set-up and Test Procedure.** The test set-up should be configured so that the specimen boundary conditions and load application simulate in-situ conditions as closely as possible. Applicable guidelines are given in CoLA/UCI Committee (1999) Section 3, ASTM (1995a) Section 5, and ASTM (1995b) Section 14. The test procedure should follow the loading history recommendations given in this report. No specific recommendations on loading rate are given here, but reference is made to ISO (1999) which recommends a displacement rate between 0.1 and 10 mm/sec.

**Instrumentation.** Measurement should be made of all force and deformation parameters that significantly affect specimen behavior and are needed to evaluate and quantify important failure modes. Pertinent guidelines are given in ATC (1992) Section 3.4, CoLA/UCI Committee (1999) Section 3, and ASTM (1995a) Section 6.

### 1.4 Documentation of Specimens and Test Results

Experiments should be documented to the extent that an individual not involved in the testing program can carry out an objective interpretation of test results, considering all variables that significantly affect specimen behavior. Specific guidelines for documentation are given in ATC (1992) Section 5, and CoLA/UCI Committee (1999) Section 4. The following summary recommendations are taken primarily from ATC-24 (ATC 1992).

For each experiment the following information should be documented:

1. Geometric data and important details of the test specimen, including fabrication/construction details (including joining and hold-downs), boundary conditions, constraints, and applied loads (e.g., location and magnitude of gravity loading).
2. Locations of instruments for the measurement of primary response parameters (parameters needed to evaluate the performance of the test specimen).
3. All material test data needed for performance evaluation.
4. The following data for the force and deformation control parameters. [The terminology used here is that employed in ATC-24.]
   - A schematics of the deformation control history with sequential cycle numbers indicated at the positive peaks.
   - A trace of the force-deformation history that shows all important aspects of the response, including points of maximum response and other important points that are needed to define salient hysteretic characteristics of the specimen.
   - The deformation (force) value(s) that have been used for deformation (force) control.
   - Numerical values of the following measurements for the positive and negative excursions of individual cycles, with appropriate sign. An excursion is part of a cycle and extends from zero force to maximum deformation to zero force (see Figure 1 of ATC-24 for definitions of force and deformation parameters). Only those data points that show an appreciable change compared to previously registered values need to be documented.
     - Peak deformation, $\delta_{i+}$ and $\delta_i$.
     - Deformation at start of excursion, $\delta_{0,i+}$ and $\delta_{0,i}$.
     - Measured plastic deformation range, $(\Delta\delta_{pm})_{i+}$ and $(\Delta\delta_{pm})_i$.
     - Force at peak deformation, $Q_{i+}$ and $Q_i$.
     - Maximum force in excursion, $Q_{max,i+}$ and $Q_{max,i}$.
     - Force in primary excursion at the peak deformation of the previous primary excursion.
     - Slope of $Q-\delta$ diagram at start of loading, $K_{0,i+}$ and $K_{0,i}$.
     - Slope of $Q-\delta$ diagram at start of unloading $K_{i+}$ and $K_i$.
     - Area enclosed by $Q-\delta$ diagram of excursion, $A_{i+}$ and $A_i$.

5. Observations made during the test and identification of any problems that may affect the interpretation of the data.

6. Data similar to those listed under 4. should be documented for other response parameters to the extent needed for a performance evaluation.

No documentation is requested at this time for elastic stiffness and yield force, because for timber structures these quantities are not yet well defined and numerical values depend on the definition used.
2. Testing Protocol for Force Controlled Quasi-Static Cyclic Testing

2.1 Loading History for Force Controlled Quasi-Static Cyclic Testing

The following loading history should be applied to components whose behavior is controlled by forces rather than deformations (see Fig. 6):

- Five cycles* with an amplitude of 0.5Q₀
- Five cycles with an amplitude of 0.7Q₀
- A primary cycle with an amplitude of 0.8Q₀
- Two trailing cycles (amplitude = 0.6Q₀)
- A primary cycle with an amplitude of 0.9Q₀
- Two trailing cycles (amplitude = 0.675Q₀)
- A primary cycle with an amplitude of 0.9Q₀
- Two trailing cycles (amplitude = 0.675Q₀)
- A primary cycle with an amplitude of 1.0Q₀
- Two trailing cycles (amplitude = 0.75Q₀)
- A primary cycle with an amplitude of 1.0Q₀
- Two trailing cycles (amplitude = 0.75Q₀)
- Additional steps of the same pattern with an increase in force amplitude of 0.1Q₀, i.e., two sequences of one primary cycle of amplitude equal to that of the previous primary cycle plus 0.1Q₀, followed by two trailing cycles of amplitude equal to 0.75 times that of the last primary cycle.

* For components that do not undergo load reversals (e.g., anchor bolts), the term “cycle” refers to a half cycle from zero load to maximum load followed by unloading.

Reference force Q₀:

This force is the maximum force the test specimen is expected to experience in the maximum considered earthquake, and assuming that the proposed basic loading history has been applied to the test specimen. It is a measure of the force capacity of the specimen. It will be necessary to estimate this force capacity prior to the test. This estimate can be based on previous experience or the execution of a monotonic test.

Additional considerations:

1. The test should be continued in the predetermined pattern until the maximum load applied in a primary cycle can no longer be increased by the increment 0.1Q₀.
2. A final definition of acceptable performance is not provided here. Testing and analysis will tell what acceptability criteria should be used. A basic concept is to define the force level associated with acceptable performance as that primary cycle amplitude at which both of the following criteria are fulfilled for the last time:

   (a) The two sequences of one primary cycle followed by two trailing cycles can be executed without brittle failure.
(b) After the last trailing cycle the specimen can still be loaded to a predetermined fraction $\eta$ of the maximum force amplitude (point C in Fig. 6). For reporting of CUREE/Caltech Woodframe test results a value of $\eta = 0.8$ is recommended.

3. The force level associated with acceptable performance does not have to be equal to the selected value of $Q_0$. It may be associated with a smaller or larger deformation amplitude.

2.2 Specimen Fabrication, Testing, Instrumentation, and Documentation

A near-fault loading history is not provided for force-controlled testing. The reason is that the force demands under near-fault ground motions will not differ much from those under ordinary ground motions, and the cumulative damage effects will be smaller. In all respects, except loading history, a force controlled cyclic test should follow the guidelines provided in Sections 1.3 and 1.4 for deformation controlled cyclic testing.

3. Time Histories for Shaking Table Testing

Time histories should be applied in a manner that permits evaluation of the performance of wooden houses at various performance levels. The performance levels of interest may be taken from guidelines such as FEMA 273 or SEAOC Vision 2000. They are associated with ground motions of specific return periods. Widely used return periods are 2475, 475, 72, and 43 years.

The following time history records are recommended to simulate seismic conditions at various return periods:

At return periods of 475 years and smaller:

A typical ordinary (not near-fault) record that represents, in shape, the NEHRP design spectrum for soil type D in the period range of interest (from about 0.1 to 1.0 sec.). The preferred choice is the Northridge 94 Canoga Park record. The acceleration response spectra of the two horizontal components of this record are shown in Fig. 7. An alternative choice is the Loma Prieta 89 Hollister Differential Array record.

- Whenever feasible the two horizontal components of the record should be applied simultaneously (preferable together with the vertical components).
- If only one component can be applied, then the larger of the two components should be used.
- Scale factors for the ground motions should be determined such that the acceleration response spectrum of the larger of the two horizontal components provides, in the vicinity of the fundamental period of the structure to be tested, a reasonable match with the NEHRP soil type D spectrum for the appropriate return period.
- The NEHRP soil type D spectra for the selected return periods need to be determined first. The basis for these spectra are USGS hazard curves for spectral acceleration at periods of 0.2 and 1.0 seconds, which are used to construct the constant acceleration and constant velocity branches of the NEHRP spectra. The spectral accelerations should be taken directly from USGS maps for the appropriate location and return period. If maps for selected return
periods are not available, then the 475 year return period map should be taken as the basis, and scale factors for spectral accelerations, $S_f$, should be determined in accordance with the following equation proposed in FEMA 273:

$$S_f = \left( \frac{\text{return period}}{475} \right)^{0.44} \quad (1)$$

- Since the so determined scaled spectra are for USGS site category $S_B/S_C$, the spectral values for appropriate return periods should be modified to soil type D conditions by using the NEHRP’97 $F_a$ and $F_v$ site coefficients. This provides the spectral amplitudes and shapes that should be used to scale the ground motion record.

At very long return periods (in the order of 2500 years):

At very long return periods it is appropriate to assume that the seismic hazard in an area like Los Angeles is controlled by fault ruptures close to the site. Thus, a near-fault record should be used to simulate seismic conditions associated with this hazard. The preferred choice is the Northridge 94 Rinaldi Receiving Station record. The acceleration response spectra of the two horizontal components of this record are shown in Fig. 8. An alternative choice is the Kobe 95 Takatori Station record.

- Whenever feasible, the two horizontal components of the record should be applied simultaneously (preferable together with the vertical components).
- If only one component can be applied, then the fault-normal component should be used.
- The near-fault record should not be scaled because insufficient knowledge exists at this time to scale near-fault records to return period specific hazard levels.
PROPOSED LOADING HISTORY

Ordinary Ground Motions

Figure 1 Loading History for Basic Cyclic Load Test
Figure 2  Definition of $\Delta_m$ and its Relation to a Cyclic Test
PROPOSED LOADING HISTORY -- Abbreviated
Ordinary Ground Motions

Figure 3  Abbreviated Loading History for Basic Cyclic Load Test
PROPOSED LOADING HISTORY -- Simplified
Ordinary Ground Motions

Figure 4  Simplified Loading History for Basic Cyclic Load Test
Figure 5 Loading History for Near-Fault Cyclic Load Test
PROPOSED LOADING HISTORY -- FORCE CONTROL

Figure 6  Loading History for Force Controlled Cyclic Load Test
ELASTIC STRENGTH DEMAND SPECTRA
NR94cnp (Two Horizontal Components) $\xi = 5\%$

Figure 7 Acceleration (Strength Demand) Spectra for Northridge 94 Canoga Park Record (Recommended Choice for Ordinary Record)

Elastic SDOF Strength Demands
NR94rrs, $\xi = 5\%$

Figure 8 Acceleration (Strength Demand) Spectra for Northridge 94 Rinaldi Receiving Station Record (Recommended Choice for Near-Fault Record)
Figure 9  Example Application of Basic Loading History (UCSD Woodframe Testing Program)
Commentary to Proposed Testing Protocols

C1. Considerations in Development of Loading Protocols

C1.1 Considerations for Deformation-Controlled Quasi-Static Cyclic Testing

It is recognized that quasi-static cyclic load testing may be performed for a variety of objectives, and that the objective should drive the loading history to be applied to a test specimen. Basic objectives of experimentation may be as follows:

- Provide knowledge on cyclic behavior characteristics useful for
  - analytical modeling
  - improvement of detailing
  - development of design equations
- Permit acquisition and documentation of consistent data that can be used for performance assessment at various performance levels and for comparison of performance of different components tested in different laboratories.

Each objective may provide strong arguments for the adoption of different loading protocols. It can be argued that the applied loading history should follow behavior and should not be predetermined at all, because behavior is the unknown quantity that necessitates testing. It can be argued further that a single loading protocol cannot cover all components, behavior modes, and failure modes that may be experienced by woodframe structures. These are strong arguments that have prevailed for many years. However, recently it has been recognized that the need exists to share experimental results on a worldwide basis to facilitate advancement of knowledge. Moreover, international trade agreements make it necessary to establish protocols for testing that can be employed universally for performance assessment that is independent of subjective opinions and national codes or guidelines. Modern information technology, which so much facilitates sharing of data, has given another large boost to the need for consistency in data in order to make experiments useful to more than the one who has performed them.

The one aspect that makes consistent interpretation of data of past experiments a difficult task is the fact that the performance of components under cyclic loading is history dependent, because of cumulative damage considerations. If this were not the case, any test to very large deformations would suffice, even a monotonic one. Because of the dependence on “loading” (more realistically, “deformation”) history, a pattern needs to be established that permits consistent interpretation of data, preferably for all the aforementioned objectives.

With this preamble in mind, as well as with the recognition that no one history can serve all purposes, the following criteria are used as the background for the development of the loading protocols presented here:

- Cumulative damage concepts should be considered to the extent necessary to make testing representative for seismic response behavior.
• Testing should be performed under loading histories that represent, in a conservative manner (this implies the use of 84th percentile values in most cases), the demands imposed by ground motions on structures of which the tested component (or assembly) is part.

• With the recent emphasis on performance-based design, the loading histories should be structured such that performance assessment at various performance levels is feasible. Since cumulative damage is expected to have a larger effect at low performance levels (e.g., collapse prevention), the weight in the loading history development is on the response to severe ground motions.

• The loading histories should account for earthquakes of different characteristics and return periods. The following criteria are set:
  • Emphasis should be placed on performance assessment of components at ground motion levels associated with long return periods, i.e.,
    • 10/50 events [475 years return period]
    • 2/50 events [2475 years return period]

  The records associated with these events are called “performance assessment records”.
  • It was decided to describe the 10/50 hazard by ordinary (non-near-fault) ground motions that are scaled to appropriate spectral values.
  • It was decided to describe the 2/50 hazard by near-fault ground motions.
  • Ground motions should represent conditions typical for Los Angeles conditions. [It is assumed that these ground motions are also representative for other regions of high seismicity in California.]
  • Multiple events should be considered, i.e., short return period records preceding the performance assessment records should be included.

C1.2 Considerations for Force Controlled Quasi-Static Cyclic Testing

Force controlled testing should be performed only if a suitable deformation parameter cannot be found. In general this means components whose behavior is controlled by brittle failure modes. An example is pull-out of an anchor bolt from the foundation – provided that pull-out is indeed the failure mode. Deformation controlled testing should be performed if bolt yielding is the failure mode.

Force-controlled testing has the purpose of quantifying the strength of a component. Since cumulative damage considerations may again enter, the test should be performed under a loading history that simulates that experienced by the component in an earthquake. Like in deformation controlled testing, the reference value for force application should be the maximum force the component will experience in an earthquake. All other force amplitudes should be a function of the maximum one.

The function of force controlled components is usually that of a joining or connection medium. As such, there is no need to make a distinction between the force capacity under ordinary and near-fault ground motions. Since the cumulative damage effects are larger under ordinary ground motions, it was decided to develop only one force controlled loading history and to
structure it according to the response to ordinary ground motions of the type associated with the 10/50 events identified in the previous section. Little attention needs to be paid to performance at high performance levels (e.g., continuous operation) because it can be assumed that design and behavior of a brittle component will be governed by severe ground motions.

In order to establish criteria for a force controlled loading history, response statistics must be obtained on the force amplitudes a component may experience under sets of representative ground motions. The results from the nonlinear dynamic analyses carried out for the development of the deformation controlled loading history, together with the relative deformation amplitudes of the basic deformation controlled loading history, are utilized to develop the force controlled loading history (see Section C.6.3).

C1.3 Considerations for Shaking Table Testing

Shaking table testing is an expensive proposition, even for relatively simple structural systems. In the context of the woodframe project, it is assumed that only one (or maybe two) house(s) can be tested, and that the objective is a comprehensive performance evaluation under various levels of seismic input, spanning from ground motions associated with a frequent events (say, 50/30 or 50/50), to design ground motions associated with a 10/50 event, to motions associated with a 2/50 event. However, it must be considered that damage under an earlier event may affect the dynamic response under later events, and that it is very unlikely that a wooden building will be subjected to a 10/50 event as well as a 2/50 event in its lifetime. To maximize the benefit of a shaking table testing program, it is prudent to perform tests under various levels of ground motion intensities but preserve the condition the structure is likely to be in before a severe earthquake (either a 10/50 or a 2/50 event) occurs. The latter implies the execution of shaking table test series that include the history of earthquakes the structure is likely to experience before the severe event occurs. It also implies the need for repair if a test series brings the structure to a state of damage that is not representative of realistic initial conditions.

In the selection of ground motion records the consideration of cumulative damage effects invites the selection of a sequence of records in accordance with the criteria outlined in Section C.1.1 and elaborated in Section C.2. Separate test series should be executed for the 10/50 and 2/50 performance assessments, with intermittent repair performed as needed to preserve realistic initial conditions. It is not advisable to perform an arbitrary number of small amplitude tests unless it is clear that such tests will not cause cumulative damage that inappropriately affects the failure mode in severe tests (e.g., nail fatigue versus pullout).

C.2 Selection of Ground Motion Records

Representative ground motion records are needed for analytical simulation studies on which to base the loading histories for quasi-static cyclic testing and the selection of records for shaking table tests. The following sets were developed for the purposes summarized in Section C.1:

- A set of 20 “ordinary” records (performance assessment records) representative of the 10/50 hazard level for Los Angeles conditions, on which to base the development of the
basic loading history. [“Ordinary” implies that these ground motions are recorded far enough from the fault rupture to be free of typical near-fault pulse characteristics.]

- A series of low amplitude “ordinary” records that represent small ground motions preceding the performance assessment records.
- A set of near-fault records representing rare events with a return period of 2475 years (2/50 hazard) for Los Angeles conditions. It is assumed that the 2/50 hazard is controlled by near-fault ground motions.

For all cases it is assumed that the structure is located at a site with NEHRP soil type D. Soft soil ground motions are not considered. As a starting point the USGS spectral values at various hazard levels, which were used in the SAC steel project (Somerville et al., 1997), are employed to develop target spectra for the selection and, when necessary, scaling of records. Target spectra are defined as spectra that represent the site hazard for soil type D at specified hazard levels (return periods), and within the period range of interest, which for woodframe structures is assumed to be from 0.2 to 1.0 seconds.

In the SAC steel project, USGS hazard mapping information has been used to establish anchor points for uniform hazard spectra for a typical LA site at various hazard levels (Somerville et al., 1997). The USGS spectral acceleration values at periods of 0.1, 0.2, 0.3, 0.5, and 1.0 sec. and for the 10/50 and 2/50 hazard levels are used here as starting points. These values, which come from USGS hazard maps, are for USGS site category Sb/Sc. They are modified to soil type D conditions by using the NEHRP’97 Fa and Fv site coefficients. The so obtained target spectra for the 10/50 and 2/50 hazard levels are shown in dashed lines in Fig. C.1. These spectra are obtained by fitting a curve to the five data points, which results in irregular spectral shapes. In the NEHRP’97 (IBC’2000) design approach, spectral acceleration values at 0.2 and 1.0 sec. are used as anchor points for the maximum considered earthquake (MCE) and for the design spectrum. According to (Somerville et al., 1997) the 2/50 and MCE spectral values are very close for the selected sites. Thus, a horizontal line through the 0.2 sec. value and a 1/T line through the 1.0 sec. value can be used to construct the MCE spectrum, and multiplying this spectrum by 2/3 provides the NEHRP’97 design spectrum (FEMA 302, 1997). As Fig. C.1 shows, the MCE spectrum is a good representation of the 2/50 uniform hazard spectrum, and the NEHRP design spectrum is a good representation of the 10/50 uniform hazard spectrum in the period range of primary interest.

Thus, the NEHRP design spectrum (i.e., 2/3 times the MCE spectrum) is used here as the reference spectrum for scaling of ordinary ground motions and deciding on the strength of the structural systems to be analyzed. In the subsequent analyses, all ordinary records are scaled to the spectral acceleration of the NEHRP design spectrum at the first mode period of the structure. For the later discussed near-fault ground motions this scaling process was not applied, because it is questionable whether the MCE spectrum is a reasonable representation of the 2/50 seismic hazard near a fault. For these ground motions the MCE spectrum is used only for comparison and not for scaling.
C.2.1 Set of Ordinary Records for Development of Basic Loading History

This set is assumed to be representative of the 10/50 hazard level in Los Angeles. The following criteria are employed to arrive at a representative set:

- These records are to represent “ordinary” ground motions at the design level. Ordinary implies that they are not near-fault ground motions.
- A minimum of 20 records is deemed to be necessary in order to obtain stable statistical estimates (median, 84th percentile).
- To arrive at a well defined set of records the following constraints were placed on the selection process:
  - Only California earthquakes are considered with a moment magnitude range of $6.7 \leq M_w \leq 7.3$.
  - The closest distance to the fault is bracketed between $13 \text{ km} \leq R \leq 25 \text{ km}$.
  - As much as feasible, only soil type D records are to be selected.
  - Records from several earthquakes are to be selected, without regard to the faulting mechanism.

Records were chosen from the Pacific Engineering and Analysis Strong-Motion Catalog of 10/06/98 (Silva’s database). The records come from the following five events: Superstition Hills(3), Northridge(7), Loma Prieta(6), Cape Mendocino(2) and Landers(2). From Silva’s database 23 suitable records could be found. Only 13 of these are of soil type D the other 10 are of soil type C. Three of the soil type C records were discarded based on achieving balanced contributions from different earthquakes. The elimination of these 3 records did not have a significant effect on the median and 84% spectra of the set. Random horizontal components were chosen in order to eliminate any bias in the selection process. The final set of records and their properties are listed in Table C.1.

Acceleration response (elastic strength demand) spectra for the selected unscaled 20 records are shown in Fig. C.2 together with the median and 84th percentile spectra. [Median is defined as the geometric mean (exponential of the average of the natural log values) of the data points, and the 84th percentile is defined as the median multiplied by the exponent of the standard deviation of the natural log values of data points.] The statistical spectra follow expected patterns, but show relatively small values compared to the spectrum representing the LA 10/50 hazard. Thus, scaling of records is an important issue. As discussed previously, all spectra are scaled to the 10/50 spectral acceleration value of the period of the structural model. Scaled spectra for the case of $T = 0.5 \text{ sec}$ are shown in Fig. C.3, together with the USGS 10/50 spectral target values at $T = 0.1$, 0.2, 0.3, 0.5, 1.0 and 2.0 sec. The median spectrum matches the USGS values rather well in the period range from 0.1 to 1.0 sec. It is noted that the dispersion, which is zero at $T = 0.5$, grows rapidly for periods longer than 0.5 sec. (the period elongation range) and shorter than 0.5 sec (the higher mode range, which is not relevant in this context). Because of this significant dispersion, large differences have to be expected between median and 84th percentile response values for inelastic systems.
C.2.2 Small Events Preceding a Performance Assessment Event

This issue may be important (or at least its importance needs to be evaluated) because earthquakes that will occur before the performance assessment event may cause damage whose cumulative effect may significantly alter the initial conditions (initial stiffness and state of damage) at the time of the performance assessment event. Realistic simulation of previous earthquake(s) is needed because

- disregard may underestimate the demands imposed by the performance assessment event, and
- overestimation may create an unrealistic state of damage that may misrepresent the likely mode of failure.

The following reasoning can be employed to establish a “probable” sequence (train) of ground motions (from C.A. Cornell, private communication). The question is what are reasonable representative record amplitudes for smaller events that might precede the performance assessment event at a site?

Analogous to reasoning used in, e.g., API guidelines for coincidental (secondary) load effects and in FORM-SORM “design point” definition, we ask for the conditional mean (or conditional expected) values of secondary effects given the value of the primary effect. We ask therefore, given that the structure experiences its “performance assessment event” at some point in its life, what do we “expect” has preceded it? To a first approximation, we assume it is a sequence of events of various first mode spectral acceleration levels (same spectral shape). [This reduces the problem to a scalar representation of the ground motion.]

Now we start using “expected” or “associated” arguments given the performance assessment event occurrence. We “expect” it to occur in the middle of the 50 year life, i.e., at year 25. Then we ask what is the spectral acceleration of the event we “expect” to occur once prior to the performance assessment event. The expected number of events per year is \( \lambda_x \), where \( \lambda_x \) is the mean rate of occurrence of events with \( S_a \geq x \). This rate is equal to the “annual probability” of the event. In 25 years we “expect” \( \lambda_x \cdot 25 \) such events. Setting this equal to unity, we get \( \lambda_x = \frac{1}{25} = 0.04 \), i.e., the associated event to occur once prior to the performance assessment event is one with a mean return period of \( (1/\lambda_x) \) equal to 25 years [in 50 year basis: \( p_{50} = 1 - e^{-\lambda_{50}} = 1 - e^{-50/25} = 1 - e^{-2} = 0.86 \), i.e., 86% in 50 years.]

Analogously, for the size of event we expect to occur twice before the performance assessment event, we set \( \lambda_x \cdot 25 = 2 \) and get \( \lambda_x = \frac{2}{25} = 0.08 \), i.e., a 12.5 year mean return period [in 50 year basis: \( p_{50} = 1 - e^{-\lambda_{50}} = 1 - e^{-50/12.5} = 1 - e^{-4} = 0.98 \), i.e., 98% in 50 years.]

Thus, spectral accelerations are needed at the periods of interest corresponding to return periods of 25 and 12.5 years. USGS uniform hazard curves at these return periods are not available, however, the USGS hazard values given in (Somerville et al., 1997) for the 2/50, 10/50, 50/50, and 30/50 hazards can be used to extrapolate to the referenced return periods. The extrapolated values at \( T = 0.2 \) and 1.0 sec. are 0.23g and 0.09g for the 25 year return period, and 0.14g and 0.055g for the 12.5 year return period. These values are modified for soil type D conditions with
NEHRP’97 $F_a$ and $F_v$ site coefficients, resulting in the following target spectral acceleration values:

For $T = 0.2$ sec:  
- 25 year $S_a = 0.37g$  
- 12.5 year $S_a = 0.23g$ (same for 0.5 sec.)  

For $T = 1.0$ sec:  
- 25 year $S_a = 0.21g$  
- 12.5 year $S_a = 0.13g$

It can be seen that these spectral accelerations are not negligible compared to the acceleration values for the design ground motions. However, their values are relatively small and it is reasonable to assume that their effect on initial conditions and cumulative damage will be small. Nevertheless, in all analysis cases with ordinary ground motions (10/50 hazard level) a sequence of three smaller ground motions (12.5, 25, and 12.5 years return period, respectively) precedes the ordinary ground motion that represents the 10/50 hazard level. The WN87hol (Whittier Narrows, Hollywood Storage) record is utilized to represent the 12.5 year return period, and the LV80kod (Livermore, San Ramon-Eastman Kodak) record is used to represent the 25 year return period. The records are scaled so that the spectral acceleration at the structural period is equal to the value given above.

A typical acceleration time history train (the three smaller records followed by an performance assessment record) is shown in Fig. C.4.

**C.2.3 Set of Near-Fault Records for Development of Near-Fault Loading History**

Recordings from recent earthquakes have provided much evidence that ground shaking near a fault rupture is characterized by pulses with very high energy input. This holds true particularly in the “forward” direction, where the propagation of the fault rupture towards a site at a velocity close to the shear wave velocity causes most of the seismic energy from the rupture to arrive in a single large pulse. Large pulses amplify the maximum interstory drift for elastic structures and more so for inelastic structures. Many studies are in progress on near-fault ground motion characterizations, and several studies are concerned with an evaluation of the effect of these motions on structures (Somerville et al. 1999, Krawinkler and Alavi 1998, Alavi and Krawinkler 1999).

The following observations summarize salient characteristics of near-fault ground motions in the context of this project:

- In many cases it is feasible to describe near-fault records by an equivalent square pulse of period $T_p$ and an effective velocity which is close to the PGV.
- The pulse period increases with earthquake magnitude, and the effective velocity increases with magnitude and closeness to fault.
- The response of structures is sensitive to $T/T_p$, with $T$ being the fundamental structure period.
- In particular, the response of structures with $T/T_p < 1.0$ is very different from those with $T/T_p > 1.0$. This may not be evident for elastic SDOF systems (see Fig. C.5(a)), but is clearly evident for inelastic SDOF (and MDOF) systems (see Fig. C.5(b), which shows...
displacement response histories for inelastic systems ($\mu = 6$) subjected to a near-fault record).

- Wood structures usually have short fundamental periods. Thus, the primary range of interest is $T/T_p < 1.0$.

Based on these observations, and focusing on records whose pulse period is relatively short (close to the period range of interest for woodframe structures), the six records listed in Table C.2 are selected for this study. Figure C.6 presents the acceleration, velocity, and displacement spectra of the individual records together with the median spectrum. Only the fault-normal component of the ground motion is shown and is used in the analysis. A pilot study (Alavi and Krawinkler 1999) has indicated that rotation by 45 degrees does not make the larger of the two rotated components much smaller than the fault-normal component. Thus, the fault-normal component is a reasonable representation of the larger of two orthogonal components in a random direction. The records are not scaled to a common spectral acceleration; they are used as recorded.

Figure C.6 indicates that the records are very severe. The median values are somewhat smaller than the Los Angeles MCE spectral values for periods shorter than about 0.6 sec., but they exceed the MCE values considerably for longer periods. Considering that woodframe structures have a relatively short period but are expected to undergo large inelastic deformations under these ground motions, the period elongation is expected to drive the structures into the range of very large demands.

### C.3 Selection of Structural Systems for Prediction of Response

The development of representative loading histories requires information on response behavior of representative structural systems subjected to various types of ground motions. Woodframe buildings are mostly from one to three stories high, and more often than not of one story only. The emphasis is placed here on single story buildings. Two and three story buildings will show relatively small higher mode effects and their response is not expected to differ much from that of single story buildings. The case of more than one story with a soft first story deserves special consideration. But because of its undesirable characteristics it is not considered a structure type that should control the development of representative loading histories. It is a matter of a separate demand evaluation study to assess the amplified demands for a soft story building. Basic information on demands for soft (weak) first story buildings is available in (Seneviratna and Krawinkler, 1997).

For the reasons just quoted, it was decided to focus on the prediction of response for SDOF systems and place emphasis on the type of hysteretic force-deformation behavior of the system. Baseline studies are performed with bilinear SDOF systems, but the emphasis is on hysteretic systems that are representative of woodframe buildings whose primary lateral load resistance comes from plywood shear wall panels. A literature survey shows clearly that this comprises a wide range of systems, ranging from relatively stable multi-linear systems to complex and deteriorating curvilinear systems. The database established on the University of California at Irvine CoLA test series proved to be most helpful in assessing structural behavior and deciding
on basic parameters. In the context of loading history development it serves little purpose to evaluate a large range of systems which will result in a huge variation of response parameters. There is little to be gained because for a given type of ground motion (ordinary or near-fault) a single history needs to be chosen at the end. Nevertheless, it is prudent to conduct parameter studies in order to evaluate the range of results that can be expected.

C.3.1 Common System Parameters

The period range of interest is assumed to be from 0.2 to 1.0 seconds. Specific values used in the demand prediction are 0.2, 0.35, 0.5, and 1.0 seconds. Shorter periods are conceivable (see Table C.3, taken from Foliente and Zacher, 1994), but likely will exist only for small amplitude vibrations.

It was decided to focus on relatively simple hysteretic systems that have a bilinear skeleton curve in common. For a given mass the elastic stiffness, \( K_e \), defines the period \( T \) of the system. The second slope of the bilinear skeleton is defined by a material strain hardening ratio \( \alpha = K_s/K_e \), which was selected as 0.08. This value is approximately a mean minus sigma value of judgmental slopes placed on the UCI/CoLA test results.

It is assumed that P-delta causes an effective softening of 3%, i.e., the value of \( (P\delta/h)/V \) is taken as 0.03. This value is estimated from the following approximate scenario:

- Single story building with \( T = 0.5 \) sec., \( V = 0.23W \) (corresponds to \( R = 5 \)).
- For \( V_y = 0.23W \) and \( \mu = 5 \), \( \delta_y = 7.0/5 = 1.4 \) cm
- Panel height = 8 ft = 240 cm
- \( V'/V = (P\delta/h)/V = (Wx1.4/240)/0.23W = \approx 0.03 \)

The yield strength \( V_y \) of the system is determined from the NEHRP’97 design spectrum (see Fig. C.1), by taking the elastic strength demand value, \( V_e = (S_a/g)W \), and dividing this value by a reduction factor \( R \). Thus, the strength is defined by the strength parameter \( \eta \) given as

\[
\eta = V_y/W = V_y/RW = (S_a/g)/R
\]

Global response parameters are determined for a range of \( R \) values from 1 to 6, and the response parameter of importance for the loading history development are determined for \( R \) factors of 5.0 (close to minimum required strength) and 2.0 (significant overstrength, which is present in most cases). It is important to note that the yield strength is computed with \( R \) factors that refer to the NEHRP’97 design spectrum, which is associated with the 10/50 set of ordinary records. Since all these records are scaled to a common \( S_a \) (at the first mode period), the strength value \( \eta \) associated with a given \( R \)-factor will be identical for each record. When the near-fault records are used in the analysis, the same strength values \( \eta \) are used, i.e., the relationship between \( \eta \) and \( S_a \) (i.e., the \( R \)-factor) varies between near-fault records.

A choice had to be made also on the value of equivalent damping. A viscous damping coefficient of \( \xi = 5\% \) is assumed in all cases, recognizing that this value is expected to fluctuate significantly dependent on the structural configuration.
In summary, the following system parameters are employed:

1. The structure period, $T$, is assumed to be defined by an elastic stiffness $K_e$ and the mass of the system. Values of $T = 0.2, 0.35, 0.5$, and $1.0$ are used.

2. All systems have a bilinear skeleton defined by $K_e$ and $K_s$.

3. A material strain hardening ratio of $\alpha = 0.08$ is used in all cases.

4. A P-delta softening ratio of $0.03$ is used in all cases.

5. The system yield strength is defined by the base shear coefficient $\eta = V_y/W = (S_a/g)/R$. The 10/50 values of $S_a$ are those obtained from the NEHRP’97 design spectrum.

6. Viscous damping of $\xi = 5\%$ is used in all cases.

**C.3.2 Hysteresis Models**

Even with the constraint of a bilinear skeleton, great variations of hysteretic models can be employed, which may include various options of stiffness degradation or strength deterioration. In this study the following options are considered:

1. The basic (non-degrading) bilinear model (even though rather unrealistic for wood structures) serves as a reference structural model.

2. The basic peak-oriented (Clough) model serves as an intermittent model between the bilinear model and the pinching model.

3. The basic pinching model of the type illustrated in Fig. C.7 serves as the primary model for demand evaluation. With appropriate parameters it is capable of closely simulating typical load-deformation behavior of plywood shear wall panels. Compared to the basic bilinear model the basic pinching model needs only one additional parameter, called $\kappa$, to define the point targeted after unloading. There are several variations to this model; the one implemented here is illustrated in Fig. C.7. It is noted that the value of $\kappa$ defines the pinching strength as a function of the maximum (not yield) strength in the direction of loading, and defines the pinching stiffness as a function of the maximum residual deformation in the direction of loading. In this study $\kappa$ values of $0.25$ and $0.5$ are selected. Based on the UCI/CoLA tests, a value of $\kappa = 0.5$ appears to be representative, in average, of plywood shear wall behavior.

4. Stiffness degradation and strength deterioration are phenomena that can be applied to the three basic models. Stiffness degradation may apply to the unloading stiffness or the reloading stiffness. Strength deterioration implies that the strain hardening branch of the skeleton curve moves inwards (translates towards the zero resistance axis). It also implies that this branch will degrade (the slope will become negative or smaller positive) at large deformations (or after many cycles). Any combinations of these phenomena can be considered simultaneously, and any of these phenomena can be described by a single deterioration parameter of the type (Rahnama and Krawinkler, 1993)
\[
\beta_i = \left( \frac{E_i}{E_t - \sum_{j=1}^{t} E_j} \right)^c
\]

(C.2)

in which

- \( \beta_i \) = parameter defining the deterioration in excursion \( i \)
- \( E_i \) = hysteretic energy dissipated in excursion \( i \)
- \( E_t \) = hysteretic energy dissipation capacity = \( \gamma F_y \delta_y \)
- \( \sum E_j \) = hysteretic energy dissipated in all previous excursions
- \( c \) = exponent defining the rate of deterioration

This parameter can be applied to strength deterioration or stiffness degradation, i.e.,

\[
F_j = (1 - \beta_i) F_{i-1} = \beta_j F_{i-1}
\]

or

\[
K_{u,i} = (1 - \beta_i) K_{u,i-1} = \beta_u K_{u,i-1}
\]

This model was investigated for different loading histories, and parameters were adjusted based on load-deformation data obtained from experiments (Fig. C.8). The upshot of analysis studies is that these deterioration phenomena have either benign or domineering effects, depending on the selected deterioration parameters and the intensity of the ground motion. From the perspective of loading history development, this adds another dimension that makes the choice of a representative loading history more ambiguous. The simple, and only feasible, way out is to assume that within the range of acceptable performance the specimen should sustain the imposed deformation cycles without much strength deterioration. If this is the case, then the deterioration phenomena can be ignored in the demand predictions that serve as a basis for loading history development.

C.4 Maximum Response Values

C.4.1 Results for Ordinary Ground Motions

The response of SDOF systems to the set of 20 ordinary design level ground motions is evaluated at periods of 0.2, 0.35, 0.5, and 1.0 seconds for various hysteresis systems. For this purpose the ordinary (10/50) ground motions are scaled to the target spectral acceleration values of the LA NEHRP’97 design spectrum (see Fig. C.1) at the respective periods. The scaled individual spectra and the median and 84\textsuperscript{th} percentile spectra at 0.2, 0.5, and 1.0 sec. are shown in Fig. C.9. It can be seen that the median spectra of the scaled records at \( T = 0.2 \) and \( 0.5 \) sec. are almost identical, but that the median spectrum of the scaled records at \( T = 1.0 \) sec. is substantially higher in the short period range. The reason is that the selected records have, in average, a spectral shape that has a relatively low spectral acceleration at 1.0 sec. (compared to the design spectrum). In all three cases the dispersion (which is zero at the selected periods) grows rather quickly at periods larger than the selected one. This implies that significant scatter has to be expected in the response parameters for inelastic systems.
Results for median values and 84th percentile values of selected response parameters are presented in Figs. C.10 to C.12. Most of the graphs are for the pinched hysteresis model with $\kappa = 0.5$, which is used as the primary model for demand evaluation. The graphs in Fig. C.12 show the sensitivity of the results to the type of hysteresis model.

**Strength – Ductility ($\eta – \mu$ and $R – \mu$) Graphs.** These graphs provide a picture of the strength-ductility relationships of the analyzed systems. A comprehensive graph for pinching systems ($\kappa = 0.5$) of various periods is presented in Fig. C.10. The figure shows ductility demand $\mu$ on the horizontal axis, and the strength parameter $\eta = (S_d/g)/R$ (left scale) as well as the strength reduction factor $R = (S_d/g)/\eta$ (right scale) on the vertical axis. For $\mu = 1.0$ the elastic strength demand is $\eta = (S_d/g)$ and the R-factor is 1.0. The $\eta – \mu$ curves show the increase in ductility demand with a decrease in provided strength, and the $R – \mu$ curves show the increase in ductility demand with an increase in the R-factor.

The $R-\mu$ curves show a close to linear relationship between $R$ and $\mu$ for $T = 0.5$ and 1.0 seconds. In fact the relationship is close to $R = \mu$ in the median. The slope of the $R-\mu$ curve for $T = 0.2$ sec. is much flatter, indicating a rapid increase in ductility demand with a decrease in strength. As expected, the dispersion in the results is significant, as indicated by the differences in the median and 84th percentile curves.

**Strength – Displacement ($\eta – \delta$) Graphs.** These graphs show the variation in displacement demands with a decrease in provided strength. A comprehensive graph for pinching systems ($\kappa = 0.5$) of various periods is presented in Fig. C.11. The marked data points correspond to R-factors of 1.0, 1.5, 2.0, 3.0, 4.0, 5.0, and 6.0. If the displacement curve bends to the right, the inelastic displacement is larger than the elastic one, and vice versa. For $T = 0.2$ sec. the inelastic displacements are much larger than the elastic ones (by a factor larger than 3 for $R = 6$, in the median), whereas for $T = 0.5$ and 1.0 sec. the inelastic displacements are about equal to the elastic ones (in the median). These results are expected for ordinary ground motions.

Figure C.12 illustrates the variation of displacement demands for different hysteresis models at $T = 0.2$ seconds. The pattern seen here is observed, but less pronounced, also for longer periods. The displacements increase successively as the model is changed from bilinear to Clough to pinching with $\kappa = 0.50$ to pinching with $\kappa = 0.25$. This illustrates the sensitivity to details of the hysteresis model. This sensitivity is by far the largest for $T = 0.2$, and is becoming much smaller for longer periods. For $T = 0.2$ sec. the displacement demands are smallest, but the ductility demands are largest. If ductility is the best measure for performance of woodframe structures (this is not a foregone conclusion because ductility is not well defined for wooden structures), then the $T = 0.2$ second case becomes critical. On the other hand, if displacement is the best measure, then the $T = 0.2$ sec. structure is well protected.

The $\eta-\delta$ curves facilitate visualization of the load-displacement response of the SDOF system. Connecting the origin with the top point (elastic strength demand) for each curve provides the elastic load-displacement response of the system. Horizontal lines at the various $\eta$ values (which correspond to $R = 1.5, 2, 3, 4, 5,$ and 6) provide the inelastic load-displacement response for
systems whose elastic strength demand is reduced by specific R-factors. The ductility demands can also be deduced by dividing the total displacement demands by the corresponding elastic displacement.

These graphs provide important input to the decision process for selecting representative systems for loading history development. They show that the displacement demands are strongly dependent on the period of the system. This makes the displacement capacity (rather than a relative measure such as story drift capacity) a critical parameter for the loading history, and it makes it necessary to pattern the amplitude of all other cycles in the loading history around the amplitude of the largest cycle. This is different from most loading protocols used in the past, which use a “yield” displacement (which is very difficult to define for wooden test specimens) to decide on the amplitude of cycles that ramp up to the largest cycle.

A disconcerting observation is that the analytically predicted displacement demands under the 10/50 ground motions are very large. Using the 84th percentile value for R = 5 (second lowest η value) at 0.5 sec. as a representative value, it is seen from Fig. C.11 that the displacement demand for a pinching system with κ = 0.5 is about 13 cm. Using a plywood panel of 8 ft = 240 cm, this corresponds to a drift angle of 13/240 = 0.054. This value is large already and, as will be seen later, is much exceeded by the predicted value for the 2/50 ground motions.

**Variation of Peak Parameters with Period.** For the loading history development the yield strengths corresponding to R = 5 and R = 2 are selected. For these specific R-factors, and for the pinched hysteresis model, the variation with period of maximum displacement, δmax, maximum ductility, δmax/δy, and normalized hysteretic energy dissipation (NHE = HE/(Fyδy)) is illustrated in Figs. C.13 to C.15. These figures serve as documentation of statistical values of parameters that are much needed in the decision process for the loading protocol. Values are also shown for the near-fault responses (NF). For these cases the R designation is replaced with η values since the R-factors refer to the NEHRP’97 design spectrum and not to the near-fault records.

The NHE curves are shown merely to assist in judgment. For bilinear systems the normalized hysteretic energy NHE, defined as NHE = HE/(Fyδy), is equal to the sum of all plastic deformation ranges (normalized to yield displacement) the structure experiences in the earthquake. For these cases, NHE is a direct measure of cumulative damage effects if it assumed that only inelastic excursions contribute to damage. This assumption is not made in this study, but the NHE values are useful, nevertheless, for preliminary assessment of cumulative damage effects. For instance, for R = 5 the median NHE value is much larger for T = 0.2 sec. than for T = 0.5 and 1.0 seconds. This indicates comparable cumulative damage effects for systems with T = 0.5 and 1.0, but much larger cumulative damage effects for T = 0.2 seconds.

**C.4.2 Results for Near-Fault Ground Motions**

Since these ground motions are not scaled to a common spectral acceleration, the R-factor has no meaning in this analysis. Thus, strength is referred to here by the base shear coefficient η = Vy/W only. A series of η values are chosen, somewhat arbitrarily, to establish η–δ and η–μ relationships for individual records. Graphs of medians and 84th percentile values are shown in Figs. C.16 and C.17. Because of the small sample set, the statistical values are obtained as
“counted” rather than “computed” values. As can be seen from the graphs, the inelastic displacement becomes much larger than the elastic one as the strength of the structure decreases, particularly for T = 0.2 seconds. Moreover, the displacements are very large, even for T = 0.2 seconds, and the ductility ratios take on “irrational” values for small \( \eta \) values. But, given a low strength, the large values are believed to be realistic. The reason for the extremely large displacements and ductility ratios lies in the shape of the acceleration response spectra (see Fig. C.6), which shows that the period elongation drives the short period structures into the region of high demands around 0.6 to 1.2 seconds.

The extremely large displacement demands associated with relatively low strength are disconcerting. They are also evident from Figs. C.13 and C.14, in which the maximum displacements are shown together with those for the 10/50 ordinary set of records, for the strength values associated with R = 5 and R = 2 for the NEHRP’97 design spectrum. Since these R-factors are meaningless for the near-fault records, they are replaced by the corresponding base shear strength coefficient \( \eta \). For T = 0.2 and T = 0.5 sec., the \( \eta \) values corresponding to R = 5 and 2 are 0.225 and 0.563, respectively. It can only be hoped that tests will show that the actual strength of woodframe structures is much larger than the minimum code design strength (which is in the order of \( \eta = 0.2 \)). There are good reasons to believe that this is the case because of the presence of stucco and gypsum board walls and other elements that contribute to strength and stiffness. Thus, the results for \( \eta = 0.225 \) are believed to be of academic value, and more weight in the development of the near-fault loading history is placed on the results for \( \eta = 0.563 \).

C.5 Cumulative Damage Considerations

The results discussed so far are peak values of displacement amplitudes and corresponding ductility ratios. They provide information on the demands imposed by different types of ground motions, and may serve as an anchor for the period dependent displacement capacity the test specimens should sustain in order to be acceptable according to specified performance criteria. However, they do not give insight into the loading history that should be applied in a test.

C.5.1 Cumulative Damage Issues

The response of structures to earthquake ground shaking depends on the force-deformation characteristics of its constituent components. These characteristics are defined by loading history dependent stiffness and strength properties. It is expected that for wood components (e.g., plywood wall panels) significant stiffness degradation occurs relatively early, whereas significant strength deterioration (defined here as a decrease in strength under cyclic loading compared to the strength attained under monotonic loading) occurs relatively late in the loading history. Both stiffness degradation and strength deterioration under cyclic loading are caused by damage mechanisms, which makes cumulative damage concepts the basis for the development of a representative loading history. The following general concepts/observations are employed in this development:

- Damage is cumulative and is defined, amongst others, by the full deformation range (peak to peak) of damaging excursions.
• All excursions above a certain threshold level contribute to damage and deserve consideration in the loading history development. In this work the threshold level on the excursion range is set as the smaller of $\delta_y$ or $\Delta\delta_{\text{max}}/20$, where $\Delta\delta_{\text{max}}$ is the maximum range the component is predicted to experience without significant deterioration in strength.

• The response to ground motions is random (not in a sequence of cycles of increasing or decreasing amplitude). Thus, individual excursions need to be extracted from response histories and ordered by means of a “cycle counting method”. The rainflow cycle counting method is employed (other methods will give similar results).

• Large excursions cause much larger damage than small excursions, thus, their representation in the loading history is emphasized.

• In general, the relative amount of damage caused by an excursion depends on the deformation range of the excursion, $\Delta\delta$, the mean deformation of the excursion (a measure of symmetry with respect to the undeformed configuration), and the sequence in which large and small excursions are applied to the component (sequence effects). In the response to ordinary ground motions the mean effects are usually small and are ignored.

• The number of damaging excursions, $N$, the deformation ranges of the excursions, $\Delta\delta_i$, and the sum of the deformation ranges, $\Sigma\Delta\delta_i$ are important parameters that should be simulated in the loading history.

• These parameters, and others discussed below, are obtained from statistical evaluation of the response of representative systems to sets of representative ground motions.

• Basic information is obtained by evaluating medians and 84th percentile values of the largest, second largest, third largest, etc., response excursion obtained from time history analysis and rainflow cycle counting.

For components of woodframe structures, such as plywood shear wall panels, the following additional concepts/observations are considered in the development:

• The hysteretic response of wood elements is usually of a pinched nature, as shown in the test results reproduced in Fig. 2, and is defined by the following characteristics that are illustrated in Figs. C.18 (arbitrary deformation history) and C.19 (illustration of corresponding load-deformation response):
  • Cumulative damage (measured by the energy dissipated in an excursion) is caused primarily by excursions that have a branch on the skeleton and “widen” the envelope of the load-deformation response (e.g., from origin to point 3, and from point 3 to point 11).
  • Smaller excursions, such at the one from the origin to point 1, can be considered interruptions of a subsequent larger excursion (e.g., origin to point 3).
  • A smaller excursion following a large excursion does little cumulative damage even if its range is relatively large (e.g., all excursions after point 14). Such excursions are called here “trailing” excursions.
Trailing excursions will not reach the previous peak load, because the previous peak load will only be attained (or exceeded) in an excursion that attains (or exceeds) the previous maximum amplitude.

Thus, the response history should be separated into primary excursions (those that widen the envelope [without being considered an interruption] and cause most of the damage) and trailing excursions.

All excursions occurring after the latter of the maximum positive and negative deformation peaks are trailing excursions. These excursions will do little cumulative damage. Thus, the pre-peak response history (all excursions before the latter of the maximum positive and negative peaks is reached) should be considered separately from the post-peak history.

Based on these observations it is considered important to base a representative loading history on the evaluation of primary excursions, and to have these excursions followed by smaller trailing excursions. All post-peak excursions should be given secondary consideration; they may be appended in some fashion to the loading history after the largest excursion has been executed. Thus, much attention is paid in the development of the loading history to a careful evaluation of primary excursions and to separate evaluations of all excursions for (a) the full response history, and (b) the pre-peak response history.

**Definition of Primary Excursions.** Primary excursions are excursions that widen the envelope of response in the positive or negative direction and are primary contributors to cumulative damage. They need to be represented in the loading history and are to be followed by an appropriate number of smaller trailing excursions. A positive excursion with an amplitude larger than all the preceding positive excursions is a candidate for a primary excursion. It becomes a primary excursion if it is followed by a negative excursion with an amplitude that is larger than the largest previous negative amplitude before it is followed by a positive excursion that exceeds the amplitude of the candidate excursion. If the latter occurs first, the candidate excursion is considered an interruption of the excursion with larger amplitude.

In the example illustrated in Fig. C.18, the ranges 0-3, 3-10, 10-11, and 11-14 constitute the four primary excursions of the response history. These excursions are shown in bold lines in Fig. C.19. All other excursions are trailing excursions.

**C.5.2 Process for Incorporating Cumulative Damage Effects into Loading History**

The issues outlined in the previous section drive the development of representative loading histories. The following general process is implemented to derive statistical information that can be employed to decide on number and sequence of excursions that will make up the loading history.
1. Using structural systems of various periods and hysteretic characteristics, nonlinear time history analysis is performed with the ground motion sets previously identified.

2. Three evaluations are performed. One for the entire displacement response history, one for the pre-peak response history, and one for the primary excursions alone.

3. For each time history response rainflow cycle counting is performed, which provides a series of excursions with well defined properties (deformation range and mean deformation). These excursions can be evaluated for mean and sequence effects, which is critical for near-fault responses. For the responses to ordinary ground motions the mean effects are expected to be small and are ignored. The individual excursion ranges are ordered in decreasing magnitude. Only excursions larger than the previously defined threshold value will be considered from here on.

4. For each structural system and set of ground motions, statistical values (median and 84th percentile, as defined in Section C.2.1) of the following response parameters are obtained:
   - Deformation range ($\Delta\delta_i$) and mean deformation ($\Delta\delta_{\text{mean},i}$) of each of the ordered excursions (largest, 2nd largest, 3rd largest, etc.).
   - Cumulative deformation ranges for partial sums of ranges ($\Delta\delta_{\text{max}} + \Delta\delta_2$, $\Delta\delta_{\text{max}} + \Delta\delta_2 + \Delta\delta_3$, etc., up to the sum of all damaging excursions).

5. This statistical information is presented in graphical and table forms (see Section C.6). It is employed, together with judgment that will result in conservative decisions, to develop representative loading histories.

6. A backward process is utilized to structure the loading history. This implies that the largest deformation amplitude (generically called $\Delta$) becomes the parameter on which all smaller excursions are based, i.e., all smaller excursions preceding the largest excursion are expressed as fractions of $\Delta$.

The implementation of this process is demonstrated in the next section for the basic and near-fault loading histories.

**C.6 Development of Representative Loading Histories**

Loading histories for quasi-static cyclic testing are developed based on the considerations summarized in Section C.5 and on data derived from nonlinear dynamic analyses of various SDOF systems covering a suitable range of hysteretic systems, periods, and strength values. Bilinear, peak oriented (Clough), and pinching models with $\kappa = 0.25$ and $0.5$ are used to represent structural cyclic characteristics. Systems with periods of 0.2, 0.35, 0.5, and 1.0 sec. are analyzed. Strength values that correspond to R-factors of 5 (close to code minimum strength) and 2 (considerable overstrength) are employed. Using the NEHRP “design” spectrum shown in Fig. C.1 (2/3 of NEHRP MCE spectrum), $R = 5$ results in $\eta = V_y/W = 0.225$ for $T < 0.6$ sec. and $\eta = 0.133$ for $T = 1.0$ sec., and $R = 2$ results in $\eta = 0.583$ and 0.333, respectively, for these periods.
For each case (i.e., combination of period, structural system, and strength), time history analysis is performed using the appropriate set of ground motions (ordinary records for basic history and near-fault records for near-fault history). Rainflow cycle counting is applied to all excursions and the pre-peak excursions, and the primary excursions are extracted from the response history. Pertinent results from the analysis are organized into spreadsheets, evaluated statistically (medians and 84th percentile), and plotted as needed to assist in the development of the loading histories. A representative set of data for one case (T = 0.5 sec., pinching model with $\kappa = 0.25$, and $\eta = 0.225$) is documented in Appendix A. The process of extracting information for the loading histories is discussed in the next section.

One decision had to be made up-front, based on the fact that the displacements (and ductility ratios) are strongly dependent on the selected period (and in some cases the selected strength values). This is the need to use the maximum response displacement as the anchor point for the amplitudes of individual cycles (or excursions) rather than the yield displacement. This decision is reinforced by the observation that the response of components of woodframe buildings rarely exhibits characteristics that can be associated with a yield point or, for that matter, any other break-point in the force-deformation response. It is important to point this out now because many of the decisions made later depend on this up-front decision.

C.6.1 Development of Deformation Controlled Basic Loading History

The set of 20 ordinary ground motion records (10/50 record set) is employed to derive information on which to structure the individual cycles of the basic loading history. Statistical data on the period dependence of several parameters is presented in Figs. C.20 to C.25 (for the pinching models with $\kappa = 0.5$). Figure C.20 shows the rapid increase with period of the maximum displacement range ($\Delta \delta_{\text{max}}$), whereas Fig. C.21 shows the decrease with period of the cyclic ductility ratio ($\Delta \delta_{\text{max}} / \delta_y$). The latter figures indicate that it will not be productive to use yield displacement (or ductility ratio) as an anchor point for decisions on individual cycles of the loading history.

Figure C.22 shows the variation with period and yield strength of the number of pre-peak excursions whose displacement range is larger than $\delta_y$. The significant dependence of this parameter on period and yield strength again points out that the need to make the loading history independent of yield strength. For this reason, the number of damaging excursions is also counted as the number of excursions whose range exceeds $\Delta \delta_{\text{max}} / 20$. This limit controlled the number of damaging excursions for all systems with $T = 0.5$ and 1.0 sec., whereas the first limit (excursion range > $\delta_y$) controlled for all systems with $T = 0.2$ and 0.35 seconds.

The number of primary excursions, which is shown in Fig. C.23, is small in all cases. Even for $T = 0.2$ sec. and $R = 5$, the 84th percentile value is less than 10. The conclusion is that only few of the excursions in a typical response history widen the envelope to an extent that causes significant damage, and that in all cases the number of trailing excursions is much larger than the number of primary excursions. It is judged that only about $1/6^{\text{th}}$ of all excursions are primary excursions.
The dependence of cumulative pre-peak deformation ranges ($\Sigma \Delta \delta_j/\delta_y$) on period and yield strength is also very large, see Fig. C.24, as is the dependence of the cumulative primary deformation ranges ($\Sigma \delta_{pi}/\delta_y$), see Fig. C.25. But from these two figures it is judged that the cumulative primary excursion ranges are about 1/3\textsuperscript{rd} of the total cumulative deformation ranges, rather than the 1/6\textsuperscript{th} factor judged for the number of excursions. This implies that for large excursions the difference between the number of pre-peak and primary excursions must be smaller than for small excursions. The 1/3\textsuperscript{rd} factor is confirmed from other observations and is used to structure the relative number of pre-peak and primary excursions in the basic loading history.

Basic information on the relative amplitude of excursions is derived from plots of the type presented in Figs. C.26 and C.27. They show the 84\textsuperscript{th} percentile of the magnitudes of ordered deformation ranges, obtained after rainflow cycle counting and computing statistical values of the ordered ranges of individual analysis cases. For instance, the fourth line is the 84\textsuperscript{th} percentile of the fourth largest excursion range of each analysis case of the specific system subjected to the 20 records of the 10/50 record set. Statistical values are obtained of the excursion range as well as the mean (midpoint) displacement of each excursion. The mean displacement is small for the response to ordinary records and is ignored in the development of the basic loading history; i.e., the excursion ranges are centered with respect to the zero displacement line.

Figures C.26 and C.27 show 84\textsuperscript{th} percentiles of the ranges for all excursions (pre- and post-peak) and pre-peak excursions, respectively. By superimposing the two graphs we can evaluate the relative importance of pre- and post-peak excursions. In the development of the basic loading history only the pre-peak excursions are considered explicitly. The post-peak excursions, which are believed to contribute little to cumulative damage (presuming that little to no deterioration has occurred at the end of the pre-peak history) are not considered individually in the loading history; they are lumped into the five large trailing excursions (2.5 cycles) that have to be executed after the largest primary cycle.

Information of the type presented in Fig. C.27, together with information on primary excursions and on the number of excursions is used to construct the basic loading history. The 84\textsuperscript{th} percentile values rather than median values are used in order to arrive at conservative (high) estimates of the number of excursions and their relative ranges. The relevant data (84\textsuperscript{th} percentile values) for all the cases evaluated are summarized in Tables C.4 to C.6.

Table C.4 summarizes the 84\textsuperscript{th} percentiles of the ordered pre-peak excursion ranges, normalized to the yield displacement $\delta_y$. Looking at the first row, it is evident that the normalized ranges are very much dependent on period and yield strength, with the maximum range varying from 53.9 to 4.3. The values tabulated in each column include all ranges whose magnitude is larger than the smaller of $\delta_y$ and $\Delta \delta_{\text{max}}/20$. For strong systems ($R = 2$) the number of excursions $> \delta_y$ is small (values larger than 1.0), but the number of excursions $> \Delta \delta_{\text{max}}/20$ is large. The reverse is true for weak short period systems.

A more relevant presentation of the same data is given in Table C.5. This table shows the same information, but in this case the excursion ranges are normalized to half of the 84\textsuperscript{th} percentile of the maximum range $\Delta$. The value of $\Delta$ for each case is listed in the column heading. For each
case, the individual excursion ranges as well as the cumulative excursion ranges are tabulated. When individual rows are compared across all cases, it can be seen that the large differences between cases have disappeared, particularly for the large excursion ranges (top rows), and rather uniform values of individual and cumulative ranges are obtained. These normalized ranges, together with the equivalent information for the primary ranges given in Table C.6, are used to construct the basic loading history.

The individual and cumulative ranges of the basic loading history are tabulated in the second and third column of Table C.5. An inspection of each row, all the way across from the basic loading history column to the last of the cases, provides an indication of the “goodness of fit” between the excursions of the basic loading history and the various cases investigated through time history analysis. The fit is very good at large excursions (those that contribute most to cumulative damage) and becomes less accurate as the number of excursion increases. But the concept of providing a conservative estimate of cumulative excursion ranges is maintained for most cases and most excursions.

The separation into primary and trailing excursions is accomplished by taking advantage of the information presented in Table C.6. Again, the second and third columns list the primary ranges of the proposed basic loading history, and the columns to the left list the 84th percentile values obtained from time history analysis. Only the most demanding case ($T = 0.2$ sec., $\eta = 0.225$, pinching with $\kappa = 0.25$) comes close to the demands on primary excursions imposed by the proposed loading history. Thus, conservatism is maintained.

The large number of relatively small cycles is incorporated to permit a performance evaluation at higher performance levels (e.g., immediate occupancy, continuous operation). Several of these cycles come from smaller events preceding the 2/50 events. If these smaller events can be ignored, then several of the smaller cycles can be omitted (see Section 1.1).

One can argue that the proposed loading history does not provide a close fit to any of the cases on which its derivation is based. This cannot be disputed, but lies in the nature of the problem. No single predetermined history can represent any or all realistic cases, but in a global sense and in the context of cumulative damage, the match between the proposed basic loading history and the individual cases represented by the selected ground motion records and structural systems is deemed to be satisfactory in general and in many cases actually very good.

### C.6.2 Development of Deformation Controlled Near-Fault Loading History

The development of the near-fault loading history is based mostly on the same concepts as those discussed in the previous section. The differences are in the set of selected ground motions and in the need to consider mean effects (the fact that the mean [midpoint] displacement of individual excursions is not close to zero). The pulse type near-fault ground motions will result in a response characterized by a small number of large excursions with significant mean displacement and by a large residual displacement at the end of the record.

This type of response is illustrated in Fig. C.28, which shows the time history response of a strong ($\eta = 0.563$) system subjected to three of the six selected near-fault records. A typical
force-displacement response is shown in Fig. C.29. In both figures the pulse-type nature of the response can be seen, but it is also evident that the response usually consists of more than just the single large excursion that is so often observed in the near-fault response of long period structures (Alavi and Krawinkler 1999). The reason is that for wood structures the fundamental period $T$ is usually smaller than the pulse period $T_p$. This observation complicates the development of a near-fault loading history because of the need to simulate more than a single large excursion.

Figures 30 and 31 present typical results on which the development of the near-fault loading history is based. They show medians of ordered (in magnitude) excursion ranges for all damaging excursions and pre-peak excursions. In this case median values rather than $84^{th}$ percentile values are used because the employed ground motions represent already very rare events (2/50) and the use of $84^{th}$ percentile response values is deemed to be too conservative. Because mean effects cannot be neglected in this case, the ordered ranges are shown displaced by the median of the mean (midpoint) displacement.

Information equivalent to that shown in Table C.5 for ordinary ground motions is shown in Table C.7 for near-fault ground motions. Three columns are shown for the proposed loading history and the individual cases; the individual ranges (normalized to the median of the maximum amplitude [not range]), the means (midpoints) of the ranges, and the cumulative ranges. The median of the maximum amplitude is the anchor point for the amplitudes of individual cycles.

A graphical presentation of the proposed near-fault loading history is shown in Fig. 5. Decisions on the ranges and mean values for the loading history are based on the data presented in Table C.7 and additional information on primary excursions. A relatively small number of small cycles is imposed at the beginning of the loading history because this loading history is intended only for performance evaluation at the collapse prevention level and is not intended to be applied for performance evaluation at higher performance levels.

The type of hysteretic response that will be obtained by implementing the basic and near-fault loading histories is illustrated in Figs. C.32 and C.33. These response curves are obtained by subjecting a non-deteriorating pinched hysteretic system (with $\kappa = 0.5$ and 5% strain hardening) that experiences a maximum ductility ratio of 10 to the basic loading history (Fig. C.32) and the near-fault loading history (Fig. C.33).

C.6.3 Development of Force Controlled Loading History

As mentioned in Section C.1.2, force controlled testing should be performed only if the component is expected to behave brittle and a suitable deformation parameter cannot be found. The reference value on which to base the amplitudes of individual cycles is the maximum force to which the component may be subjected in a severe earthquake. There is no compelling reason to distinguish between ordinary and near-fault ground motions, and the 10/50 set of ordinary ground motions is used as the basis for decisions on relative force amplitudes of individual cycles.
A typical force response of a pinching system with $\kappa = 0.25$ is presented in Fig. C.34. The upper graph shows the complete response history and the lower graph shows the pre-peak response history. Statistics on force level crossings obtained from time history analyses of the cases evaluated for development of the basic loading history are used to provide guidance for the selection of relative force levels. Table C.8 lists medians and 84th percentile values of level crossings at force levels of yield force $F_y$, at $0.75F_y$, and at $0.5F_y$. Values for all excursions and pre-peak excursions are tabulated. The tabulated values include all crossings, i.e., the sum of crossings in the positive and negative directions. The number of crossings was found to be rather symmetric, i.e., the number of crossings in the positive or negative direction is close to half of the tabulated values.

The tabulated level crossings are of some, but limited, value in the decision process on relative force levels for the loading history. The reason is that the number of level crossings depends strongly on the ductility demand of the system investigated. If the ductility demand is large (e.g., 21), then the maximum force for a system with 5% effective hardening is large (e.g., $2F_y$), whereas it is only $1.1F_y$ for a system with a maximum ductility of 3. For this reason the level crossings are used only as guidelines but not as a tool for final decisions.

The relative peak force values of individual cycles of the force controlled loading history shown in Fig. 6 are obtained from the abbreviated deformation controlled loading history presented in Fig. 3 and the graph shown in Fig. C.35. In this graph three bilinear systems with ductility ratios of 4, 10, and 20 and a strain hardening ratio of 0.05 are normalized to the maximum force and displacement value. Recognizing that components of wood structures usually are not of bilinear nature, curved skeleton curves are fit by judgment to the three bilinear diagrams. The envelope of these curves, together with the displacement amplitudes of the primary cycles of the basic loading history (e.g., displacement values of 0.7, 0.4, 0.3, 0.2, 0.1, 0.075, and 0.05) are used to obtain corresponding force values of primary force excursions (dashed horizontal lines). The so obtained force values are rounded up to obtain force amplitudes of the primary cycles of the force controlled loading history shown in Fig. 6. The trailing cycles are assumed to be of an amplitude of 0.75 times the preceding primary cycle. Since small cycles are believed to have negligible effects on the force capacity, all cycles with a force amplitude smaller than 0.7 (which corresponds to a displacement amplitude of 0.1) could be omitted in the loading history. Nevertheless, five cycles with force amplitude equal to $0.5Q_0$ are recommended to be executed.

**C.7 Representative Input for Shaking Table Studies**

Several ground motion records were evaluated for the purposes summarized in Section 3. The objective of record selection is to identify ground motions that are representative, in average, of the shaking a woodframe structure would experience at the hazard levels (or return periods) identified in Section 3. “In average” implies also that the records should be representative of the cumulative damage contained in the proposed loading histories.

The following time history records are recommended to simulate seismic conditions at various return periods:
At return periods of 475 years and smaller:

Select a typical ordinary (not near-fault) record that represents, in shape, the NEHRP design spectrum for soil type D in the period range of interest (from about 0.1 to 1.0 sec.). The preferred choice is the Northridge 94 Canoga Park record. The acceleration response spectra of the two horizontal components of this record are shown in Fig. C.36(a), together with the LA NEHRP design spectrum on which the development of the basic loading history is based. It can be judged that the larger of the two components may be sufficiently severe to represent the design spectrum at a period of about 0.6 seconds. For shorter periods some scaling will be necessary. If a scale factor of 1.3 is applied to the two components, the spectra shown in Fig. C.36(b) are obtained. Now the match in the short period range (less than about 0.35 sec.) is adequate, but the spectrum of the “larger” component is somewhat high from about 0.35 to 0.7 seconds.

Because of the frequency characteristics of recorded ground motions it will never be possible to obtain uniform matching for the full period range of interest and compromises will have to be made. In judging these compromises it is necessary to consider more than just the elastic shape of the spectra because of inelastic response characteristics. It would be desirable to consider the shape of inelastic spectra together with that of the elastic spectrum. For instance, the hump around the period of 0.6 sec. in the elastic spectrum of the larger component of this record will affect the inelastic response of a structure with a shorter period. In the writer’s judgment it is preferable to use recorded ground motions together with sound judgment for scaling rather than simulated ground motions whose spectrum could be matched well over a wide range of periods but whose frequency content may not be representative of realistic ground motions.

An alternative choice to the Northridge 94 Canoga Park record is the Loma Prieta 89 Hollister Differential Array record. The spectra of the two components of this record, unscaled and scaled by a factor of 1.3, are presented in Fig. C.37. Again, it is evident that good judgment will be needed to scale the record to provide the most appropriate match with the target spectrum.

At very long return periods (in the order of 2500 years):

At very long return periods it is appropriate to assume that the seismic hazard in an area like Los Angeles is controlled by fault ruptures close to the site. Thus, a near-fault record should be used to simulate seismic conditions associated with this hazard. The preferred choice is the Northridge 94 Rinaldi Receiving Station record whose acceleration, velocity, and displacement spectra are shown in Fig. C.38. The large difference in spectral shape and ordinates between the fault normal and fault parallel component is characteristic of near-fault records. An alternative to this record, although not a very desirable one because of the presence of multiple humps in the spectra, is the Kobe 95 Takatori Station record whose spectra are presented in Fig. C.39.
Acknowledgements

This work was carried out as part of the CUREE/Caltech woodframe research program. The feedback provided by many of the researchers and advisors of this program is gratefully acknowledged. In particular, much helpful feedback was provided by Professors D. Dolan, A. Filiatrault, J. Hall, R. Hanson, G. Pardoen, and C.M. Uang, and by Dr. G. Foliente and Ms. K. Cobeen. This feedback is gratefully acknowledged.
References


Table C.1 Set of 20 LA Ordinary Ground Motion Records

<table>
<thead>
<tr>
<th>Record ID</th>
<th>Earthquake Event</th>
<th>Year</th>
<th>Mw</th>
<th>Station</th>
<th>Rclosset (km)</th>
<th>Soil Type</th>
<th>Fault Mechanism</th>
<th>fHP (Hz)</th>
<th>fLP (Hz)</th>
<th>PGA (g)</th>
<th>PGV (cm/s)</th>
<th>PGD (cm)</th>
<th>Spectral Acceleration (g) ξ = 0.05</th>
</tr>
</thead>
<tbody>
<tr>
<td>SH87bra</td>
<td>Superstition Hills</td>
<td>1987</td>
<td>6.7</td>
<td>Brawley</td>
<td>18.2</td>
<td>D</td>
<td>strike-slip</td>
<td>0.13</td>
<td>20.0</td>
<td>0.116</td>
<td>17.2</td>
<td>8.6</td>
<td>0.188 0.216 0.150</td>
</tr>
<tr>
<td>SH87icc</td>
<td>Superstition Hills</td>
<td>1987</td>
<td>6.7</td>
<td>El Centro Imp. Co. Cent.</td>
<td>13.9</td>
<td>D</td>
<td>strike-slip</td>
<td>0.10</td>
<td>38.0</td>
<td>0.258</td>
<td>40.9</td>
<td>20.2</td>
<td>0.508 0.572 0.247</td>
</tr>
<tr>
<td>SH87pls</td>
<td>Superstition Hills</td>
<td>1987</td>
<td>6.7</td>
<td>Plaster City</td>
<td>21.0</td>
<td>D</td>
<td>strike-slip</td>
<td>0.20</td>
<td>18.0</td>
<td>0.186</td>
<td>20.6</td>
<td>5.4</td>
<td>0.402 0.574 0.156</td>
</tr>
<tr>
<td>NR94mal</td>
<td>Northridge</td>
<td>1994</td>
<td>6.7</td>
<td>Beverly Hills 14145 Mulhol</td>
<td>19.6</td>
<td>C</td>
<td>reverse-slip</td>
<td>0.13</td>
<td>30.0</td>
<td>0.416</td>
<td>59.0</td>
<td>13.1</td>
<td>1.015 1.248 1.020</td>
</tr>
<tr>
<td>NR94cnp</td>
<td>Northridge</td>
<td>1994</td>
<td>6.7</td>
<td>Canoga Park - Topanga Can</td>
<td>15.8</td>
<td>D</td>
<td>reverse-slip</td>
<td>0.05</td>
<td>30.0</td>
<td>0.356</td>
<td>32.1</td>
<td>9.1</td>
<td>1.104 0.931 0.291</td>
</tr>
<tr>
<td>NR94elp</td>
<td>Northridge</td>
<td>1994</td>
<td>6.7</td>
<td>Glendale - Las Palmas</td>
<td>25.4</td>
<td>D</td>
<td>reverse-slip</td>
<td>0.13</td>
<td>30.0</td>
<td>0.357</td>
<td>12.3</td>
<td>1.9</td>
<td>1.239 0.565 0.065</td>
</tr>
<tr>
<td>NR94hol</td>
<td>Northridge</td>
<td>1994</td>
<td>6.7</td>
<td>LA - Hollywood Sior FF #</td>
<td>25.5</td>
<td>D</td>
<td>reverse-slip</td>
<td>0.20</td>
<td>23.0</td>
<td>0.231</td>
<td>18.3</td>
<td>4.8</td>
<td>0.622 0.544 0.221</td>
</tr>
<tr>
<td>NR94far</td>
<td>Northridge</td>
<td>1994</td>
<td>6.7</td>
<td>LA - N Faring Rd</td>
<td>23.9</td>
<td>D</td>
<td>reverse-slip</td>
<td>0.13</td>
<td>30.0</td>
<td>0.273</td>
<td>15.8</td>
<td>3.3</td>
<td>0.742 0.401 0.153</td>
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<tr>
<td>NR94cwc</td>
<td>Northridge</td>
<td>1994</td>
<td>6.7</td>
<td>N Hollywood - Coldwater Can</td>
<td>14.6</td>
<td>C</td>
<td>reverse-slip</td>
<td>0.10</td>
<td>30.0</td>
<td>0.271</td>
<td>22.2</td>
<td>11.2</td>
<td>0.723 0.512 0.378</td>
</tr>
<tr>
<td>NR94gle</td>
<td>Northridge</td>
<td>1994</td>
<td>6.7</td>
<td>Sunland - Mt Gleason Ave</td>
<td>17.7</td>
<td>C</td>
<td>reverse-slip</td>
<td>0.05</td>
<td>30.0</td>
<td>0.157</td>
<td>14.5</td>
<td>4.3</td>
<td>0.411 0.453 0.418</td>
</tr>
<tr>
<td>LP89cap</td>
<td>Loma Prieta</td>
<td>1989</td>
<td>6.9</td>
<td>Capitola</td>
<td>14.5</td>
<td>D</td>
<td>reverse-oblique</td>
<td>0.20</td>
<td>48.0</td>
<td>0.529</td>
<td>36.5</td>
<td>9.1</td>
<td>1.347 0.805 0.456</td>
</tr>
<tr>
<td>LP89g03</td>
<td>Loma Prieta</td>
<td>1989</td>
<td>6.9</td>
<td>Gilroy Array # 3</td>
<td>14.4</td>
<td>D</td>
<td>reverse-oblique</td>
<td>0.10</td>
<td>33.0</td>
<td>0.555</td>
<td>35.7</td>
<td>8.2</td>
<td>2.032 1.064 0.267</td>
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<tr>
<td>LP89g04</td>
<td>Loma Prieta</td>
<td>1989</td>
<td>6.9</td>
<td>Gilroy Array # 4</td>
<td>16.1</td>
<td>D</td>
<td>reverse-oblique</td>
<td>0.20</td>
<td>28.0</td>
<td>0.417</td>
<td>38.8</td>
<td>7.1</td>
<td>0.635 0.953 0.288</td>
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<tr>
<td>LP89gmr</td>
<td>Loma Prieta</td>
<td>1989</td>
<td>6.9</td>
<td>Gilroy Array # 7</td>
<td>24.2</td>
<td>D</td>
<td>reverse-oblique</td>
<td>0.20</td>
<td>40.0</td>
<td>0.226</td>
<td>16.4</td>
<td>2.5</td>
<td>0.707 0.519 0.108</td>
</tr>
<tr>
<td>LP89hda</td>
<td>Loma Prieta</td>
<td>1989</td>
<td>6.9</td>
<td>Hollister Diff. Array</td>
<td>25.8</td>
<td>D</td>
<td>reverse-oblique</td>
<td>0.10</td>
<td>33.0</td>
<td>0.279</td>
<td>35.6</td>
<td>13.1</td>
<td>0.472 0.920 0.548</td>
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<tr>
<td>LP89wvc</td>
<td>Loma Prieta</td>
<td>1989</td>
<td>6.9</td>
<td>Saratoga - W Valley Coll.</td>
<td>13.7</td>
<td>C</td>
<td>reverse-oblique</td>
<td>0.10</td>
<td>49.0</td>
<td>0.332</td>
<td>61.5</td>
<td>36.4</td>
<td>0.639 0.627 0.650</td>
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<td>CM92for</td>
<td>Cape Mendocino</td>
<td>1992</td>
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<td>Fortuna - Fortuna Blvd #</td>
<td>23.6</td>
<td>C</td>
<td>reverse-slip</td>
<td>0.07</td>
<td>23.0</td>
<td>0.116</td>
<td>30.0</td>
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<td>0.216 0.280 0.180</td>
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<td>CM92rno</td>
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<td>7.1</td>
<td>Rio Dell Overpass - FF #</td>
<td>18.5</td>
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<td>23.0</td>
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<td>1992</td>
<td>7.3</td>
<td>Desert Hot Springs #</td>
<td>23.3</td>
<td>C</td>
<td>strike-slip</td>
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<td>23.0</td>
<td>0.154</td>
<td>20.9</td>
<td>7.8</td>
<td>0.399 0.297 0.341</td>
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<td>1992</td>
<td>7.3</td>
<td>Yermo Fire Station #</td>
<td>24.9</td>
<td>D</td>
<td>strike-slip</td>
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<td>23.0</td>
<td>0.152</td>
<td>29.7</td>
<td>24.7</td>
<td>0.441 0.439 0.326</td>
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</table>
Table C.2. Identification and Properties of Selected Near-Fault Records

<table>
<thead>
<tr>
<th>Designation</th>
<th>Earthquake</th>
<th>Station</th>
<th>Magnitude</th>
<th>Distance</th>
<th>Tp (sec)</th>
<th>PGV (cm/sec)</th>
</tr>
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<tr>
<td>LP89lex</td>
<td>Loma Prieta, 1989</td>
<td>Lexington</td>
<td>7.0</td>
<td>6.3</td>
<td>1.0</td>
<td>179</td>
</tr>
<tr>
<td>NR94rrs</td>
<td>Northridge, 1994</td>
<td>Rinaldi</td>
<td>6.7</td>
<td>7.5</td>
<td>1.0</td>
<td>174</td>
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<tr>
<td>NR94newh</td>
<td>Northridge, 1994</td>
<td>Newhall</td>
<td>6.7</td>
<td>7.1</td>
<td>1.3</td>
<td>119</td>
</tr>
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<td>KB95kobj</td>
<td>Kobe, 1995</td>
<td>JMA</td>
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<td>0.6</td>
<td>0.9</td>
<td>160</td>
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<tr>
<td>KB95tato</td>
<td>Kobe, 1995</td>
<td>Takatori</td>
<td>6.9</td>
<td>1.5</td>
<td>2.0</td>
<td>174</td>
</tr>
<tr>
<td>MH84cyld</td>
<td>Morgan Hill, 1984</td>
<td>Coyote</td>
<td>6.2</td>
<td>0.1</td>
<td>0.8</td>
<td>65</td>
</tr>
</tbody>
</table>

Table C.3. Summary of natural periods and frequencies of wood and wood-based buildings from experiments and calculation estimates [Foliente, G.C. and E.G. Zacher (1994)]

<table>
<thead>
<tr>
<th>Building Type</th>
<th>Natural Period Tn (sec)</th>
<th>Natural Frequency (1/Tn) (Hz)</th>
<th>Reference(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>One- and two-story New Zealand residential</td>
<td>0.1 to 0.6</td>
<td>1.7 to 10.0</td>
<td>(24)</td>
</tr>
<tr>
<td>One-story truss-frame residential</td>
<td>0.14 to 0.26</td>
<td>3.8 to 7.2</td>
<td>(33)</td>
</tr>
<tr>
<td>Two- and three-story N. American residential</td>
<td>0.14 to 0.32</td>
<td>3.0 to 7.0</td>
<td>(62)</td>
</tr>
<tr>
<td>Two-story residential (Greece)</td>
<td>0.18 to 0.22</td>
<td>4.5 to 5.6</td>
<td>(67)</td>
</tr>
<tr>
<td>Two-story base-isolated residential</td>
<td>0.48 to 1.25</td>
<td>0.8 to 2.1</td>
<td>(55)</td>
</tr>
<tr>
<td>One-, one and a half-, and two-story N. American residential and school buildings</td>
<td>0.06 to 0.25</td>
<td>4.0 to 18.0</td>
<td>(61)</td>
</tr>
<tr>
<td>One-, two- and three-story Japanese residential</td>
<td>0.11 to 0.33</td>
<td>3.0 to 9.0</td>
<td>(3)</td>
</tr>
<tr>
<td>Three-story Japanese residential</td>
<td>0.16 to 0.20</td>
<td>4.7 to 6.2</td>
<td>(51, 70)</td>
</tr>
<tr>
<td>One-, and two-story commercial/industrial (plywood roof diaphragm and concrete/masonry walls)</td>
<td>0.20 to 0.80</td>
<td>1.2 to 5.1</td>
<td>(11)</td>
</tr>
<tr>
<td><strong>Range of Values</strong>*</td>
<td><strong>0.06 to 0.80</strong></td>
<td><strong>1.2 to 18.0</strong></td>
<td></td>
</tr>
</tbody>
</table>

* Excluding the two-story base-isolated building.
<table>
<thead>
<tr>
<th>R = 2</th>
<th>R = 5, ( \eta = 0.25 )</th>
<th>( y = 0.225 ) cm Pre-Peak Excursions</th>
<th>Normalized by ( \delta_t )</th>
<th>Ordinary Ground Motions</th>
<th>( y = 0.225 ) cm</th>
<th>( y = 0.225 ) cm</th>
<th>( y = 0.225 ) cm</th>
<th>( y = 0.225 ) cm</th>
<th>( y = 0.225 ) cm</th>
<th>( y = 0.225 ) cm</th>
<th>( y = 0.225 ) cm</th>
<th>( y = 0.225 ) cm</th>
<th>( y = 0.225 ) cm</th>
<th>( y = 0.225 ) cm</th>
<th>( y = 0.225 ) cm</th>
<th>( y = 0.225 ) cm</th>
<th>( y = 0.225 ) cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table C.4 84th Percentile Values of Ordered Pre-Peak Excursions (Normalized by ( \delta_t )), Ordinary Ground Motions</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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</table>
### Table C.5 84th Percentile Values of Ordered Pre-Peak Excursions Normalized by $\Delta = \text{Max. Range}/2$ - Ordinary Ground Motions

| Number | Leading History for Ground Motions | H = 0.025 | H = 0.05 | H = 0.10 | H = 0.20 | H = 0.40 | H = 0.80 | H = 1.00 | H = 1.20 | H = 1.40 | H = 1.60 | H = 1.80 | H = 2.00 | H = 2.20 | H = 2.40 | H = 2.60 | H = 2.80 | H = 3.00 | H = 3.20 | H = 3.40 | H = 3.60 | H = 3.80 | H = 4.00 | H = 4.20 | H = 4.40 | H = 4.60 | H = 4.80 | H = 5.00 | H = 5.20 | H = 5.40 | H = 5.60 | H = 5.80 | H = 6.00 | H = 6.20 | H = 6.40 | H = 6.60 | H = 6.80 | H = 7.00 | H = 7.20 | H = 7.40 | H = 7.60 | H = 7.80 |
|-------|-----------------------------------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|
### Table C.6 84th Percentile Values of Ordered Primary Excursions, Ordinary Ground Motions

**Ordinary Ground Motions**

**Ordered Primary Excursions - 84th Percentile Values**

Normalized to half of computed 84th percentile maximum range, $\Delta$

<table>
<thead>
<tr>
<th>Excursion Number</th>
<th>Proposed Loading History</th>
<th>$\eta$ (Normalized by $\eta = 0.225$)</th>
<th>Pinching, $\alpha = 3%$</th>
<th>Pinching, $\alpha = 8%$</th>
<th>Pinching, $\alpha = 25%$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.5000</td>
<td>0.3500</td>
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</tr>
<tr>
<td>2</td>
<td>1.2000</td>
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<tr>
<td>3</td>
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<td>0.3500</td>
<td>0.2500</td>
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<td>0.2500</td>
</tr>
<tr>
<td>4</td>
<td>0.7000</td>
<td>0.3500</td>
<td>0.3000</td>
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</tr>
<tr>
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<td>0.4000</td>
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<tr>
<td>8</td>
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<td>0.3500</td>
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<td>0.3500</td>
<td>0.5000</td>
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### Table C.7 Median Values of Ordered Pre-Peak Excursions (Normalized by $\Delta = \Delta$ = Max. Amplitude), Near-Fault Ground Motions

**Near-Fault Ground Motions**

**Ordered Pre-Peak Excursions - Median Values**

Normalized to median of the maximum amplitude, $\Delta$

<table>
<thead>
<tr>
<th>Excursion Number</th>
<th>Proposed Loading History</th>
<th>$\eta$ (Normalized by $\eta = 0.225$)</th>
<th>Pinching, $\alpha = 3%$</th>
<th>Pinching, $\alpha = 8%$</th>
<th>Pinching, $\alpha = 25%$</th>
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<tr>
<td>1</td>
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<tr>
<td>2</td>
<td>1.2000</td>
<td>0.6000</td>
<td>0.1500</td>
<td>0.1500</td>
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<td>3</td>
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Table C.8 Force Level Crossings, Ordinary Ground Motions

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<th>84th %</th>
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<td>28.2455</td>
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<tr>
<td>0.75 Fy, All Excursions</td>
<td>36.0195</td>
<td>57.1176</td>
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<tr>
<td>Fy, Pre-Peak</td>
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<td>35.2123</td>
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<tr>
<td>0.5 Fy, Pre-Peak</td>
<td>19.0224</td>
<td>29.1881</td>
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<td>29.8533</td>
<td>49.0564</td>
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<tr>
<td>Fy, Pre-Peak</td>
<td>20.0316</td>
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<tr>
<td>0.5 Fy, Pre-Peak</td>
<td>9.9403</td>
<td>9.9348</td>
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<th>84th %</th>
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<td>0.5 Fy, Pre-Peak</td>
<td>9.9403</td>
<td>9.9348</td>
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Pinching, $\kappa = 0.25$

Bilinear Clough Pinching, $\kappa = 0.5$

Pinching, $\kappa = 0.25$

Pinching, $\kappa = 0.5$

Pinching, $\kappa = 0.25$

Pinching, $\kappa = 0.5$

$\delta_y = 0.224$ cm

$\delta_y = 0.685$ cm

$\delta_y = 1.398$ cm

$\delta_y = 3.315$ cm

$\delta_y = 0.559$ cm

$\delta_y = 3.495$ cm
USGS/NEHRP Response Spectra  Soil Type D

Figure C.1  Target Spectra for Scaling of Ground Motion Records

ELASTIC STRENGTH DEMAND SPECTRA
Set of 20 Unscaled Records $\xi=5\%$

Figure C.2  Spectra of Unscaled Ordinary records
ELASTIC STRENGTH DEMAND SPECTRA
Set of 20 records $\xi=5\%$ Scaled to USGS LA10/50 $S_a(0.5)$

Figure C.3  Spectra for Ordinary Records, Scaled to 10/50 $S_a$ at $T = 0.5$ sec.

Ground Motion Time History -Northridge-Canoga Park (NOR3) scaled to design $S_a$ at $T=0.5$ s., with small events

Figure C.4  Acceleration Time History of Record Train
(a) Elastic Response

(b) Response of Inelastic Systems with $\mu = 6$

Figure C.5 Near-Fault Response Histories for SDOF Systems; Northridge Rinaldi Receiving Station
Figure C.6 Acceleration, Velocity, and Displacement Spectra of Fault-Normal Component of Selected Near-Fault Records
Fig. C.7 Pinching Hysteresis Model

Plywood Wall – Test

Plywood Wall - Simulation

Figure C.8. Experimentally Obtained and Simulated Hysteretic Responses
For Period T = 0.2 sec.

For Period T = 0.5 sec.

For Period T = 1.0 sec.

Figure C.9 Strength Demand (Acceleration) Spectra Scaled to NEHRP’97 Design Values

Strength vs. Ductility, All Exc.; Pinching Model
LA 10/50; $\xi=5\%$; $\alpha=8\%$; $P_{\Delta}=-3\%$; $\kappa=0.5$

Figure C.10 Strength – Ductility ($\eta - \mu$ and $R - \mu$) Curves, Pinching with $\kappa = 0.5$
Figure C.11  Strength – Displacement (\( \eta \) – \( \delta \)) Curves for Different Periods, Pinching with \( \kappa = 0.5 \)

Figure C.12  Strength – Displacement (\( \eta \) – \( \delta \)) Curves for Different Hysteresis Models
Maximum Displacement vs. Period; Pinching Model
LA 10/50 & N-R; ξ = 5%; α = 8%; PΔ = -3%; κ = 0.5

Figure C.13  Variation of Maximum Displacement with Period, R = 5 and 2

Maximum Ductility vs. Period; Pinching Model
LA 10/50 & NR; ξ = 5%; α = 8%; PΔ = -3%; κ = 0.5

Figure C.14  Variation of Maximum Ductility with Period, R = 5 and 2

Normalized Hysteretic Energy vs. Period; Pinching Model
LA 10/50; ξ = 5%; α = 8%; PΔ = -3%; κ = 0.5

Figure C.15  Variation of Normalized Hysteretic Energy Dissipation with Period, R = 5 and 2
Str. vs Disp.(counted) - All Exc.; Pinching Model
Near-Fault; $\zeta=5\%, \alpha=8\%, P_{\Delta}=-3\%, \kappa=0.5$

![Figure C.16 Strength – Displacement ($\eta$ – $\delta$) Curves for Near-Fault Responses](image)

Str. vs Duct.(counted) - All Exc.; Pinching Model
Near-Fault; $\zeta=5\%, \alpha=8\%, P_{\Delta}=-3\%, \kappa=0.5$

![Figure C.17 Strength – Ductility ($\eta$ – $\mu$) Curves for Near-Fault Responses](image)
Figure C.18  Arbitrary Deformation History

Figure C.19  Hysteresis Response for Deformation History of Fig. C.18
Maximum Displacement Range vs. Period; Pinching Model
LA 10/50; $\xi=5\%$; $\alpha=8\%$; $P\Delta=-3\%$; $\kappa=0.5$

![Graph of Maximum Displacement Range vs. Period]

Figure C.20  Variation of Max. Displacement Range with Period, Pinching Model, $\kappa = 0.5$

Maximum Cyclic Ductility vs. Period; Pinching Model
LA 10/50; $\xi=5\%$; $\alpha=8\%$; $P\Delta=-3\%$; $\kappa=0.5$

![Graph of Maximum Cyclic Ductility vs. Period]

Figure C.21  Variation of Cyclic Ductility with Period, Pinching Model, $\kappa = 0.5$
Figure C.22 Variation of Number of Pre-Peak Excursions $\delta y$ with Period, Pinching Model, $\kappa = 0.5$

Figure C.23 Variation of Number of Primary Excursions with Period, Pinching Model, $\kappa = 0.5$
Normal. Cum. Excursion Range $>\delta_y$ vs. Period, Pre-Peak
Pinching Model; LA 10/50; $\xi=5\%$; $\alpha=8\%$; $P\Delta=-3\%$; $\kappa=0.5$

Figure C.24 Variation of Cumulative Pre-Peak Excursion Ranges with Period, Pinching Model, $\kappa = 0.5$

Norm. Cum. Primary Excursion Ranges vs. Period
Pinching Model; LA 10/50; $\xi=5\%$; $\alpha=8\%$; $P\Delta=-3\%$; $\kappa=0.5$

Figure C.25 Variation of Cumulative Primary Excursion Ranges with Period, Pinching Model, $\kappa = 0.5$
84th % Centered Def. Ranges, All Exc. T=0.5s, Pinching M.
LA 10/50 ; R=5 (θ=0.22) ; δy=1.398cm; ξ=5%; α=8%; P-Δ= -3% ; κ=0.25

Figure C.26 84th Percentile Values of Ordered Deformation Ranges, All Excursions, Ordinary Records, T = 0.5 sec., Pinching Model, κ = 0.5

84th % Centered Def. Ranges Pre-Peak Ex. T=0.5s, Pinching M.
LA 10/50; R=5 (θ=0.22) ; δy=1.398cm; ξ=5%; α=8%; P-Δ= -3% ; κ=0.25

Figure C.27 84th Percentile Values of Ordered Deformation Ranges, Pre-Peak Excursions, Ordinary Records, T = 0.5 sec., Pinching Model, κ = 0.5
Figure C.28  Response Time Histories for Pinching System $(T = 0.5, \eta = 0.563, \kappa = 0.5)$ 
Subjected to Three Near-Fault Records
Force-Def. Response - All Exc., NR94newh; T=0.5 s Pinching M.
Nr-Flt, \( R = 2 \ (\eta = 0.563); \delta y = 3.39\text{cm}; \zeta = 5\%, \alpha = 8\%, \, P_\Delta = -3\%, \, \kappa = 0.5 \)

Figure C.29  Typical Force-Displacement Response to Near-Fault Time History
Median Def. Ranges (counted), All Exc. T=0.5s, Pinching M.
Near-Fault (η=0.225) ; \( \delta y = 1.356 \text{cm} ; \xi = 5\% ; \alpha = 8\% ; P\Delta = -3\% ; \kappa = 0.5 \)

Figure C.30  Median Values of Ordered Deformation Ranges, All Excursions,
Near-Fault Records, T = 0.5 sec., \( \eta = 0.225 \), Pinching Model, \( \kappa = 0.5 \)

Median Def. Ranges(counted), Pre-Peak. T=0.5s, Pinching M.
Near-Fault (η=0.225) ; \( \delta y = 1.356 \text{cm} ; \xi = 5\% ; \alpha = 8\% ; P\Delta = -3\% ; \kappa = 0.5 \)

Figure C.31  Median Values of Ordered Deformation Ranges, Pre-Peak Excursions,
Near-Fault Records, T = 0.5 sec., \( \eta = 0.225 \), Pinching Model, \( \kappa = 0.5 \)
Figure C.32  Simulation of Basic Loading History with Nondeteriorating Pinched Hysteresis Model

Figure C.33  Simulation of Near-Fault Loading History with Nondeteriorating Pinched Hysteresis Model
Figure C.34 Representative Force History for an Ordinary Record
Figure C.35  Estimation of Force Amplitudes from Deformation Amplitudes of Basic Loading History
Figure C.36  Recommended Choice for Ordinary Record, Northridge 94, Canoga Park Record
ELASTIC STRENGTH DEMAND SPECTRA
LP89hda (Two Horizontal Components) $\xi = 5\%$

Figure C.37  Recommended Alternate Choice for Ordinary Record, Loma Prieta 89, Hollister Differential Array Record
Figure C.38  Recommended Choice for Near-Fault Record, Northridge 94, Rinaldi Receiving Station Record
Figure C.39  Recommended Alternate Choice for Near-Fault Record, Kobe 95, Takatori Station
Appendix A – Representative Results for Response to Ordinary Ground Motion

Case: $T = 0.5$ sec., Pinching Model with $\kappa = 0.25$, $R = 5$ ($\eta = 0.225$)

Def. Resp. Time Hist., all Exc., NOR3; $T=0.5s$; Pinching Mod.
LA 10/50; $R=5(\eta=0.22)$; $\delta y=1.398cm$; $\xi=5\%$; $\alpha=8\%$; $P\Delta=-3\%$; $\kappa=0.25$

Def. Resp. Time Hist., Pre-Peak Exc., NOR3; $T=0.5$ s; Pinch. M.
LA 10/50; $R=5(\eta=0.22)$; $\delta y=1.398cm$; $\xi=5\%$; $\alpha=8\%$; $P\Delta=-3\%$; $\kappa=0.25$
Force-Def. Response; All Exc., NOR3 ; T=0.5 s. Pinching Mod.
LA 10/50; R=5 (η=0.22); δy=1.398cm; ξ=5%; α=8%; PΔ=-3%; κ=0.25

Force-Def. Response; PrePeak., NOR3 ; T=0.5 s. Pinching Mod.
LA 10/50; R=5 (η=0.22); δy=1.398cm; ξ=5%; α=8%; PΔ=-3%; κ=0.25
Rainflow Excursions, all Exc., NOR3; T=0.5 s; Pinching Model
LA 10/50; R=5 (\(\eta=0.22\)); \(\delta y=1.398\text{cm}\); \(\xi=5\%\); \(\alpha=8\%\); \(P\Delta=-3\%\); \(\kappa=0.25\)

Rainflow Excursions, Pre-Peak Ex., NOR3; T=0.5 s; Pinching M.
LA 10/50; R=5 (\(\eta=0.22\)); \(\delta y=1.398\text{cm}\); \(\xi=5\%\); \(\alpha=8\%\); \(P\Delta=-3\%\); \(\kappa=0.25\)
Median Deformation Ranges, All Exc. T=0.5s, Pinching M.
LA 10/50; R=5 (η=0.22) ; δy=1.398cm ; ξ=5% ; α=8% ; PΔ= -3%; κ=0.25

Median Deformation Ranges Pre-Peak Exc. T=0.5s, Pinching M.
LA 10/50; R=5 (η=0.22) ; δy=1.398cm; ξ=5%; α=8%; PΔ=-3%; κ=0.25
Median Centered Def. Ranges, All Exc.; T=0.5s; M. Pinching
LA 10/50 ;R=5 (\eta=0.22) ; \delta_y=1.398cm; \xi=5%; \alpha=8%; P\Delta= -3%; \kappa=0.25

Median Centered Def. Ranges Pre-Peak Ex. T=0.5s, M. Pinching
LA 10/50 ;R=5 (\eta=0.22) ; \delta_y=1.398cm ; \xi=5% ; \alpha=8% ; P\Delta= -3% ; \kappa=0.25
84th % Centered Def. Ranges, All Exc. T=0.5s, Pinching M.
LA 10/50 ; R=5 (η=0.22) ; δy=1.398cm; ξ=5%; α=8%; P-Δ= -3% ; κ=0.25

84th % Centered Def. Ranges Pre-Peak Ex. T=0.5s, Pinching M.
LA 10/50; R=5 (η=0.22) ; δy=1.398cm; ξ=5%; α=8%; PΔ=-3%; κ=0.25
CDF of Deform. Ranges, all Exc. T=0.5s Pinching Model
LA 10/50; R=5 (η=0.22); δy=1.398cm; ξ=5%; α=8%; PΔ=-3%; κ=0.25

CDF of Deform. Ranges, Pre-Peak Ex. T=0.5s Pinching Model
LA 10/50; R=5 (η=0.22); δy=1.398cm; ξ=5%; α=8%; PΔ=-3%; κ=0.25
Pinching Model - T=0.5 s - All Exc.
LA 10/50; k=0.25; P-D=3%; a=8%; R=5 (η=0.22); dy=1.398 cm

Normalized Deformation Ranges

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 |

83

Pinching Model - T=0.5 s - All Exc.
LA 10/50; k=0.25; P-D=3%; a=8%; R=5 (η=0.22); dy=1.398 cm

Normalized Deformation Ranges

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 |

83

Pinching Model - T=0.5 s - All Exc.
LA 10/50; k=0.25; P-D=3%; a=8%; R=5 (η=0.22); dy=1.398 cm

Normalized Deformation Ranges

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 |

83
Pinching Model Modified -T=0.5 s. - Pre-Peak Exc.
LA 10/50; η=0.25; P_∆x=3%; η=8%; R=5 (η=0.22); δy=1.396 cm
Normalized Deformation Ranges

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Number of excursions

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| Sum of Normalized deformation ranges

<p>| | | | | | | | | | | | | | | | | | | | | | | |</p>
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84
### Normalized Deformation Ranges from Primary Excursions for each History; T=0.5 s.; Pinching Model

**LA 10/50; Rs5 (η=0.22), ηy=1.398 cm; ηx=5%; P=3%; ω=0.25**

| cm1 | cm2 | lan1 | lan2 | lnp | lnp2 | lnp3 | lnp4 | lnp5 | lnp6 | nor1 | nor2 | nor3 | nor4 | nor5 | nor6 | nor7 | nor8 | nor9 | nor10 | sup1 | sup2 | sup3 |
|-----|-----|------|------|-----|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|
| 9   | 9.166 | 3.021 | 3.599 | 4.118 |
| 9   | 7.837 | 2.779 | 3.860 |
| 10  | 6.424 | 3.037 |
| 11  | 5.065 | 2.748 |
| 12  | 3.175 |
| 13  | 2.748 |

**Sum norm**


**Sum**

| 56.984 | 47.950 | 246.644 | 101.045 | 122.127 | 34.365 | 23.656 | 18.182 | 40.460 | 84.696 | 52.542 | 34.436 | 13.572 | 98.941 | 31.889 | 95.735 | 37.410 | 99.572 | 20.810 | 22.970 |

### Deformation Ranges of Primary Excursions; T=0.5 s.; Pinching Model

**LA 10/50; Rs5 (η=0.22), ηy=1.398 cm; ηx=5%; P=3%; ω=0.25**

<table>
<thead>
<tr>
<th>Num</th>
<th>ln</th>
<th>Median</th>
<th>STDlog</th>
<th>94%</th>
<th>99%</th>
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<tr>
<td>n</td>
<td>3.592</td>
<td>34.889</td>
<td>0.747</td>
<td>73.636</td>
<td></td>
</tr>
<tr>
<td>Sum norm</td>
<td>3.887</td>
<td>48.782</td>
<td>0.747</td>
<td>102.958</td>
<td></td>
</tr>
</tbody>
</table>

**Mean-ln Median STDlog 94% 99%**

| n   | 3.592 | 34.889 | 0.747 | 73.636 |
| Sum norm | 3.887 | 48.782 | 0.747 | 102.958 |

**Sum = sum of primary excursion ranges in a response time history**

### Normalized Deformation Ranges of Primary Excursions for each History; T=0.5 s.; Pinching Model

**LA 10/50; Rs5 (η=0.22), ηy=1.398 cm; ηx=5%; P=3%; ω=0.25**

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<thead>
<tr>
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<th>cm2</th>
<th>lan1</th>
<th>lan2</th>
<th>lnp</th>
<th>lnp2</th>
<th>lnp3</th>
<th>lnp4</th>
<th>lnp5</th>
<th>lnp6</th>
<th>nor1</th>
<th>nor2</th>
<th>nor3</th>
<th>nor4</th>
<th>nor5</th>
<th>nor6</th>
<th>nor7</th>
<th>nor8</th>
<th>nor9</th>
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85
Deformation Ranges for RainFlow Cycle Counting in Pinching Model

LA 10/50; R=5 (η=0.22), δy=1.398 cm; ξ=5%; α=8%; P∆=3%; κ=0.25

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<th>σ_ln</th>
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<th>Median</th>
<th>84th</th>
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<td>(cm)</td>
<td>(cm)</td>
<td>Normaliz</td>
<td>(cm)</td>
<td>Normaliz</td>
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<tr>
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Deformation Ranges of Primary Excursions, T=0.5 s.; Pinching Model

LA 10/50; R=5 (η=0.22), δy=1.398 cm; ξ=5%; α=8%; P∆=3%; κ=0.25

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<th>Median</th>
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</table>
Force Statistics (level crossings); All exc. T=0.5 s Pinching Model
LA 10/50; R=5\(\eta=0.22\); \(\delta y=1.398\) cm; \(\xi=5\%\); \(\alpha=8\%\); \(P-\Delta=-3\%\); \(k=0.25\); \(F_y=220.79\) (kg-cm/sec²/kg)

<table>
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</tr>
<tr>
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</tr>
<tr>
<td>Neg Py 10</td>
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</tr>
<tr>
<td>Max</td>
<td>10 9 8 7 6</td>
</tr>
<tr>
<td>0.5 s</td>
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<tr>
<td>Pos Py 10</td>
<td>5 4 3 2 1</td>
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<td>Neg Py 10</td>
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<tr>
<td>Max</td>
<td>10 9 8 7 6</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Force Value</th>
<th>Max Absolute</th>
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</thead>
<tbody>
<tr>
<td>Pos Py</td>
<td>294.60 311.47 370.17 305.21 313.55 242.08 240.87 242.64 276.60 360.57 395.95 276.99 239.61 287.51 264.24 312.08 294.75 236.55 265.67 249.74 283.307 322.733</td>
</tr>
<tr>
<td>Neg Py</td>
<td>3.923 6.440</td>
</tr>
<tr>
<td>Max</td>
<td>15.109 21.180</td>
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</tbody>
</table>

Force Statistics (level crossings); Pre-Peak. T=0.5 s Pinching Model
LA 10/50; R=5\(\eta=0.22\); \(\delta y=1.398\) cm; \(\xi=5\%\); \(\alpha=8\%\); \(P-\Delta=-3\%\); \(k=0.25\); \(F_y=220.79\) (kg-cm/sec²/kg)

<table>
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<th>Force Amplitude</th>
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<tbody>
<tr>
<td>0.75 s</td>
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<td>Max</td>
<td>15.109 21.180</td>
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Force Statistics (level crossings); Pre-Peak. T=0.5 s Pinching Model
LA 10/50; R=5\(\eta=0.22\); \(\delta y=1.398\) cm; \(\xi=5\%\); \(\alpha=8\%\); \(P-\Delta=-3\%\); \(k=0.25\); \(F_y=220.79\) (kg-cm/sec²/kg)

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<td>10 9 8 7 6</td>
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<tr>
<td>Neg Py</td>
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</tr>
<tr>
<td>Max</td>
<td>15.109 21.180</td>
</tr>
</tbody>
</table>

Force Value

Max Force

| Pos Py      | 294.60 311.47 370.17 305.21 313.55 242.08 240.87 242.64 276.60 360.57 395.95 276.99 239.61 287.51 264.24 312.08 294.75 236.55 265.67 249.74 283.307 322.733 |
| Neg Py      | 3.923 6.440 |
| Max         | 15.109 21.180 |

87