A Computer Program for Seismic Analysis of 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; John Coil and James Russell, Assistant Managers
4. **Economic Aspects**: Tom Tobin, Tobin Associates, Manager
5. **Education and Outreach**: Jill Andrews, Southern California Earthquake Center, Manager
The Testing and Analysis Element of the CUREE-Caltech Woodframe Project consists of 23 different investigations carried out by 16 different organizations (13 universities, three consulting engineering firms). This tabulation includes an independent but closely coordinated project conducted at the University of British Columbia under separate funding than that which the Federal Emergency Management Agency (FEMA) has provided to the Woodframe Project. Approximately half the total $6.9 million budget of the CUREE-Caltech Woodframe Project is devoted to its Testing and Analysis tasks, which is the primary source of new knowledge developed in the Project.

### Woodframe Project Testing and Analysis Investigations

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Two-Story House (design) |
| 1.1.2 | Khalid Mosalam, Stephen Mahin, UC Berkeley  
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| 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


ACKNOWLEDGEMENTS

The research project described in this report was funded by the Consortium of Universities for Earthquake Engineering (CUREE) as part of the CUREE-Caltech Woodframe Project (“Earthquake Hazard Mitigation of Woodframe Construction”), under a grant administered by the California Office of Emergency Services and funded by the Federal Emergency Management Agency.

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

The main objective of this research project is the formulation and development of a simple numerical model to predict the dynamic characteristics, quasi-static pushover and seismic response of woodframe buildings. In this model the building structure is composed of two primary components: rigid horizontal diaphragms and nonlinear lateral load resisting shear wall elements. The actual three-dimensional building is degenerated into a two-dimensional planar model using zero-height shear wall spring elements connected between the diaphragms and the foundation. The pinched, strength and stiffness degrading hysteretic behavior of each wood shear wall in the building can be characterized using an associated numerical model that predicts the wall’s load-displacement response under general quasi-static cyclic loading. In turn, in this model, the hysteretic behavior of each shear wall is represented by an equivalent nonlinear shear spring element. With this simple approach, the response of the building is defined in terms of only three-degrees-of-freedom per floor. This numerical model has been incorporated into the computer program SAWS: Seismic Analysis of Woodframe Structures. The predictive capabilities of this program are compared with recent shake table tests performed on a full-scale two-story woodframe house.
REPORT LAYOUT

Part 1: NONLINEAR ANALYSIS OF WOODFRAME BUILDINGS – MODEL FORMULATION

Section 1-1 gives a brief overview of the state of the research in numerically modeling the dynamic response of woodframed buildings. Section 1-2 describes the underlying theory leading to the numerical formulation of the SAWS computer program. Section 1-3 presents the solution strategies used in the SAWS program to perform a static pushover analysis, determine the dynamic characteristics of the structure and perform a dynamic analysis under seismic input. Section 1-4 provides concluding comments on the development of the SAWS model. Part 1 concludes with a reference section of cited work.

Part 2: NONLINEAR ANALYSIS OF WOODFRAME BUILDINGS – MODEL VERIFICATION

Section 2-1 gives a brief overview of past research conducted on testing and numerically modeling the structural response of woodframed buildings. Section 2-2 describes the CUREE-Caltech Woodframe Project shake table test structure that is used to validate the SAWS computer program. Section 2-3 describes the modeling and calibration procedures that were utilized to analyze the shake table test structure using the SAWS computer program. Section 2-4 presents static pushover and dynamic response predictions of the SAWS model for the CUREE-Caltech Woodframe Project shake table test structure and compares these against the test results. Section 2-5 provides concluding comments on the implementation and verification of the SAWS program. Part 2 concludes with a reference section of cited work.
Part 3: SAWS PROGRAM USER MANUAL

Section 3-1 gives an overview of the SAWS computer program. The specifications and limitations of the SAWS program are delineated in Section 3-2. General comments on creating an input data file for the SAWS program are given in Section 3-3. Detailed instructions for creation of the input data file are presented in Section 3-4. Part 3 concludes with a sample input data file and summary output file.
PART 1

NONLINEAR ANALYSIS OF WOODFRAME BUILDINGS – MODEL FORMULATION
1-1. INTRODUCTION

Light-frame residential wood buildings are by far the most common structures constructed in North America. These building systems range in size from small, single-story dwellings to large, multi-level, multiple-occupancy condominiums and apartments. The primary structural components of these building are typically horizontal floor diaphragms; horizontal or sloped roof diaphragms and vertical shear walls. Each of these structural elements is generally composed of sawn lumber framing members attached to sheathing panels using nails and/or adhesives. Typically, these elements are interconnected through nailed connections to make up the overall structural system for the building. Whereas the construction of these structural components has been standardized and design procedures codified, structural analysis tools to evaluate their full performance are limited in number and general availability. In part, this dearth of analysis tools can be attributed to the complexity of wood-based structural components. As a construction material, wood is inhomogeneous, anisotropic and exhibits relatively large variability in its mechanical properties. Sheathing-to-framing connection response is also generally nonlinear, exhibiting pinching behavior and strength and stiffness degradation under cyclic loading. In addition, these built-up components have a relatively high degree of structural redundancy, which adds to the complexity of their modeling. Consequently it is not surprising, to some degree, that the behavior of these structural components have largely been identified through testing programs. A recent comprehensive review summarizes experimental and analytical work that has been conducted on the performance of wood-based structural subassemblies (Yancy et al 1998).

At the building level, the complexity and redundancy in the structural system can be greatly accentuated. As a result, full three-dimensional structural analysis models for wood framed buildings are few. It has been reported (Itani and Cheung 1983) that as late as 1983 there
was no analytical model dedicated to the three-dimensional analysis of woodframe houses. Covering the period up to 1998, the above-cited comprehensive review lists the development of only six 3-D models for woodframe structures over that time span (Gupta and Kuo 1987; Schmidt and Moody 1989; Yoon and Gupta 1991; Ge 1991; Kasal et al. 1994; Tarabai and Itani 1997). Of these, only one model is capable of performing a dynamic analysis (Tarabai and Itani 1997). This is in stark contrast to the abundance of static and dynamic structural analysis tools available to evaluate the response of building frames composed of structural steel or reinforced concrete.

It has been noted (Kasal et al. 1994) that if all of the structural detail within a woodframe building is captured within a finite element model, that the required computational overhead can overwhelm practical computing capability. Therefore, model reduction techniques must be implemented to produce usable numerical models. To varying degrees this approach was adopted in all of the above cited 3-D structural analysis models developed for woodframe buildings. Further to this, the more detailed the model the greater the dependency on test data to calibrate the model. Unfortunately, the requisite test data is not always readily available.

This study presents a simple, versatile numerical model that predicts the dynamic characteristics, quasi-static pushover and seismic response of light-frame wood buildings. With this model, the actual three-dimensional building is degenerated into a two-dimensional planar model composed of zero-height shear wall spring elements connecting the floor and roof diaphragms together or to the foundation. All diaphragms in the building model are assumed to have infinite in-plane stiffness. Using this simple modeling approach, the response of the building is defined in terms of only three-degrees-of-freedom (3-DOF) per floor. Each shear wall spring element is calibrated to reflect the pinched, strength and stiffness degrading hysteretic behavior of the shear wall it is modeling in the structure. Requisite test data is limited to that necessary to
characterize the behavior of the shear walls. This 3-D numerical model has been incorporated into the computer program SAWS: Seismic Analysis of Woodframe Structures. In Part 2 of this report the predictive capabilities of this model are compared with recent shake table tests performed on a full-scale two-story woodframe house. It is shown that the model predictions are in good agreement with the test results with respect to both the dynamic characteristics and seismic response of the building structure.

1-2. NUMERICAL MODEL FORMULATION

1-2.1. Structural Configuration of Building Model

For illustrative purposes, the structural configuration of a typical woodframe building will be presented in terms of the simplified single-story building layout shown in Fig. 1-1. The main structural elements that compose the building are: exterior and/or interior shear walls, interior partition walls and the floor and/or roof diaphragms. The building structure is attached to a rigid foundation. In the modeling of the structure, it is assumed that both the floor and roof elements have sufficiently high in-plane stiffness to be considered as rigid elements. This is expected to be a reasonable assumption for typically constructed diaphragms with a planar aspect ratio of the order of 2:1, as supported by experimental results from full-scale diaphragms tests (Philips et al. 1990).
Figure 1-1: Components of a Single-Story Woodframe Structure.

Figure 1-2: Model of the Single-Story Woodframe Structure.
Further to this, it is assumed that an equivalent horizontal diaphragm can model a sloping roof. As a final modeling step, the 3-D building structure is degenerated to a planar model by assigning zero-height to all of the shear wall elements that connect the horizontal diaphragms to the foundation. Each shear wall in the building structure is, in turn, represented by an equivalent single-degree-of-freedom (SDOF) shear spring element via a calibration procedure discussed in a subsequent section. Applying this overall modeling approach to the building structure illustrated in Fig. 1-1 produces the building model shown in Fig. 1-2.

### 1-2.2. Kinematic Assumptions for the Building Model

To analyze the building model presented in Fig. 1-2 a global rectangular coordinate system is prescribed as shown so that the entire building structure is contained within the first quadrant. The global degrees-of-freedom of each diaphragm are then assigned at the origin of the coordinate system. Under the assumption that each diaphragm is rigid, only three global degrees-of-freedom are required to completely describe its rigid body motion: two translations $U$ and $V$ and one rotation $\Theta$. Under a general displaced state of the diaphragm, specified by values of $U$, $V$ and $\Theta$, the resulting displacement of any point $p$ on the diaphragm is given by:

$$\begin{align*}
    u_p &= U + x_p [\cos \Theta - 1] - y_p \sin \Theta \\
    v_p &= V + y_p [\cos \Theta - 1] + x_p \sin \Theta \\
    \theta_p &= \Theta
\end{align*}$$

(1)

Limiting the diaphragm to only small rotations, (1) reduces to the following linearized kinematic relationships, which can be expressed conveniently in matrix form:

$$\begin{bmatrix}
    u_p \\
    v_p \\
    \theta_p
\end{bmatrix} =
\begin{bmatrix}
    1 & 0 & -y_p \\
    0 & 1 & x_p \\
    0 & 0 & 1
\end{bmatrix}
\begin{bmatrix}
    U \\
    V \\
    \Theta
\end{bmatrix} = \Lambda D$$

(2)
where $\Lambda$ is a transformation matrix and $\mathbf{D} = [\mathbf{U}, \mathbf{V}, \theta]^T$ is the global displacement vector.

1-2.3. Load Vector and Stiffness Matrix Formulation

Consider the application of external forces $f_{px}$ and $f_{py}$ located at any point $p$ on the diaphragm, with the line of action of $f_{px}$ parallel to the x-axis and $f_{py}$ parallel to the y-axis. These forces are statically equivalent to the following system of forces applied at the origin:

\[
\begin{bmatrix}
F_U \\
F_V \\
M_\theta
\end{bmatrix} =
\begin{bmatrix}
1 & 0 & 0 \\
0 & 1 & 0 \\
-y_p & x_p & 1
\end{bmatrix}
\begin{bmatrix}
f_{px} \\
f_{py}
\end{bmatrix} = \Lambda^T \begin{bmatrix}
f_{px} \\
f_{py}
\end{bmatrix}
\]

where $\mathbf{F} = [F_U, F_V, M_\theta]^T$ is the global force vector that is conjugate to the global displacement vector $\mathbf{D}$. The force transformation relationship given by (3) can be used to represent all types of loads applied to the diaphragm including inertia forces as well as the restoring forces developed in the SDOF shear elements.

For a given building structure let $N_x$ and $N_y$ denote the number of SDOF shear elements parallel to the x-axis and y-axis, respectively. For the example building model shown in Fig. 1-2 $N_x = N_y = 3$. For the i-th SDOF shear element aligned parallel to the x-axis let $k_{ix}$ denote its in-plane stiffness. Similarly, let $k_{iy}$ denote the in-plane stiffness of the j-th SDOF shear element aligned parallel to the y-axis. Out-of-plane stiffness in all shear elements is assumed to be zero. This assumption is expected to be reasonable for an isolated shear wall, however it also implies that intersecting shear walls behave independently of each other, which typically is not the case (Groom and Leichti 1994).

The force developed in a SDOF shear element resulting from a displacement of the diaphragm can be related to the global forces in the structure through (3). Also, the displacement
of the SDOF shear element can be related to the global displacement of the diaphragm through (2). Applying and combining these two steps over all SDOF shear elements in the building model results in the generation of the global stiffness matrix $K$:

$$K = \sum_{ix=1}^{Nx} (\Lambda_{ix}^T K_{ix} \Lambda_{ix}) + \sum_{jy=1}^{Ny} (\Lambda_{jy}^T K_{jy} \Lambda_{jy})$$

(4)

where $K_{ix}$ and $K_{jy}$ are given, respectively, by:

$$K_{ix} = \begin{bmatrix} k_{ix} & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix} \quad \text{and} \quad K_{jy} = \begin{bmatrix} 0 & 0 & 0 \\ 0 & k_{jy} & 0 \\ 0 & 0 & 0 \end{bmatrix}$$

(5a,b)

As presented, $K_{ix}$ and $K_{jy}$ apply to a 3-DOF building model such as shown in Fig. 1-2. In (4), the coordinates in the transformation matrices $\Lambda_{ix}$ and $\Lambda_{jy}$ identify the point of attachment of the SDOF shear element to the diaphragm. Extension of the formulation of the global force vector and stiffness matrix to multi-story building structures is straightforward.

In the global stiffness matrix $K$ presented above it is important to note that the contributing stiffness of each SDOF shear element is, for the problem at hand, load (or displacement) history dependent. A general characterization of the hysteretic response of the SDOF shear elements is presented in the subsequent section.

### 1-2.4. SDOF Hysteretic Model of Wood Shear Walls

The force-deformation response of a wood shear wall with nailed sheathing-to-framing connectors is nonlinear under monotonic loading and additionally exhibits pinched hysteretic behavior with strength and stiffness degradation under general cyclic loading. It is well recognized that the global force-deformation response of a wood shear wall is qualitatively very
similar to that of the individual sheathing-to-framing connectors used in the construction of the wall (Dolan and Madsen 1992; Folz and Filiatrault 2001). Consequently, when properly calibrated, the same hysteretic model used for the sheathing-to-framing connectors can be applied to model the global wall response. The authors have previously developed a general hysteretic model for sheathing-to-framing connectors which is defined in terms of a number of path following rules to reproduce the response of a connector under arbitrary cyclic loading (Folz and Filiatrault 2001). This same hysteretic model is adopted herein to model the global cyclic response of a wood shear wall.

Figure 1-3 shows the assumed force-deformation behavior of a wood shear wall under monotonic loading to failure. The monotonic racking response, in terms of top-of-wall force \( F \) and displacement \( \delta \) (see the insert in Fig. 1-3) is modeled by the following nonlinear relationship:

\[
F = \begin{cases} 
\text{sgn}(\delta) \cdot (F_0 + r_1 K_0 |\delta|) \left[ 1 - \exp \left( -\frac{K_0 |\delta|}{F_0} \right) \right], & |\delta| \leq \delta_u \\
\text{sgn}(\delta) \cdot F_u + r_2 K_0 [\delta - \text{sgn}(\delta) \cdot \delta_u], & |\delta_u| < |\delta| \leq |\delta_F| \\
0, & |\delta| > |\delta_F| 
\end{cases}
\]

This force-deformation model is characterized by six physically identifiable parameters: \( F_0, K_0, r_1, r_2, \delta_u \) and \( \delta_F \). Phenomenologically, (6) captures the crushing of the framing members and sheathing along with yielding of the connectors. Beyond the displacement \( \delta_u \), which is associated with the ultimate load \( F_u \), the load-carrying capacity is reduced. Failure of the wall under monotonic loading occurs at the displacement \( \delta_F \).

Next, consider the force-deformation response of a shear wall under the cyclic loading shown as an insert in Fig. 1-4. The basic path following rules which define the hysteretic model are identified and briefly discussed. In Fig. 1-4 force-deformation paths OA and CD follow the monotonic envelope curve as expressed by (6). All other paths are assumed to exhibit a linear
Figure 1-3: Force-Displacement Response of a Wood Shear Wall under Monotonic Loading. Hysteretic Model is Fitted to Test Data for a 2.4 m x 2.4 m Shear Wall with 9.5 mm Thick OSB Panels (Durham 1998). The Insert Shows the Racking Deformation Mode of the Shear Wall.

relationship between force and deformation. Unloading off the envelope curve follows a path such as AB with stiffness \( r_3 K_0 \). Here the wall unloads elastically. Under continued unloading the response moves onto path BC, which has reduced stiffness \( r_4 K_0 \). The very low stiffness along this path exemplifies the pinched hysteretic response displayed by wood shear walls under cyclic loading. This behavior occurs because of the previously induced crushing of the framing members and sheathing panels around the connectors (in this case as the wall followed the path
OA). Loading in the opposite direction for the first time forces the response onto the envelope curve CD. Unloading off this curve is assumed elastic along path DE, followed by a pinched response along path EF, which passes through the zero-displacement intercept \( F_i \), with slope \( r_4K_0 \).

Continued re-loading follows path FG with degrading stiffness \( K_p \), as given by

\[
K_p = K_0 \left( \frac{\delta_0}{\delta_{\text{max}}} \right)^{\alpha}
\]

(7)
with $\delta_0 = \left(\frac{F_0}{K_0}\right)$ and $\alpha$ a hysteretic model parameter which determines the degree of stiffness degradation. Note from (7) that $K_p$ is a function of the previous loading history through the last unloading displacement $\delta_{un}$ off the envelope curve (corresponding to point A in Fig. 1-4), so that

$$\delta_{max} = \beta \delta_{un}$$

(8)

where $\beta$ is another hysteretic model parameter. A consequence of this stiffness degradation is that it also produces strength degradation in the response. If on another cycle, the shear wall is displaced to $\delta_{un}$, then the corresponding force will be less than $F_{un}$ which was previously achieved. This strength degradation is shown in Fig. 1-4 by comparing the force levels obtained at points A and G. Also, with this model under continued cycling to the same displacement level, the force and energy dissipated per cycle is assumed to stabilize.

In total, 10 parameters are required to define this hysteretic model. To obtain these parameters for a particular wood shear wall an analysis tool such as the CASHEW (Cyclic Analysis of SHEar Walls) program (Folz and Filiatrault 2000) can be employed. CASHEW assumes that a shear wall is composed of pin-connected rigid framing members, elastic shear deformable sheathing members and nonlinear sheathing-to-framing connectors that follow the hysteretic model described above. With the CASHEW program, a given wall is first subjected to the CUREE-Caltech testing protocol (Krawinkler et al. 2000); after which a fitting procedure extracts the parameters to represent the wall response by an equivalent SDOF shear element. This model reduction approach has been successfully evaluated against experimental tests. As an example, the force-deformation response graph of Fig. 1-4 was produced using the parameters obtained from the CASHEW program, given in Table 1-1, for a 2.4 m x 2.4 m shearwall with 9.5 mm thick oriented strand board (OSB) sheathing panels that was also tested cyclically and under
TABLE 1-1: Fitted Hysteretic Parameters for the SDOF Shear Element Model of a 2.4 m x 2.4 m Wood Shear Wall with 9.5 mm Thick OSB Sheathing Panels.

<table>
<thead>
<tr>
<th>$K_0$ (kN/mm)</th>
<th>$r_1$</th>
<th>$r_2$</th>
<th>$r_3$</th>
<th>$r_4$</th>
<th>$F_0$ (kN)</th>
<th>$F_I$ (kN)</th>
<th>$\Delta_u$ (mm)</th>
<th>$\alpha$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.44</td>
<td>0.081</td>
<td>-0.022</td>
<td>1.31</td>
<td>0.064</td>
<td>15.1</td>
<td>3.13</td>
<td>60.0</td>
<td>0.74</td>
<td>1.10</td>
</tr>
</tbody>
</table>

an earthquake ground motion on a shake table (Durham 1998). Comparison between the experimental results and the predicted responses of the reduced shear wall model showed good agreement (Folz and Filiatrault 2001).

In recent years, there has been a proliferation of cyclic tests performed on shear walls. This database of experimental results can also be used to obtain equivalent SDOF shear elements for each of the shear walls that have been tested. For example, Fig. 1-5a shows the cyclic response of a 2.4 m x 2.4 m shear wall with 22 mm thick stucco that was tested at the University of California at Irvine for the City of Los Angeles (COLA) Project (Pardoen 2000). For this test the wall was subjected to the sequential phase displacement protocol (Porter 1987). Figure 1-5b shows the response of the fitted SDOF shear element and Table 1-2 lists the associated model parameters. Comparing the two figures shows reasonably good reproduction by the simple calibrated SDOF shear element. This type of information could be used to study the lateral stiffness and strength contributions of stucco at the structural system level by using these calibrated shear wall spring elements in a building model.
Figure 1-5: Cyclic Response of a 2.4 m x 2.4 m Shear Wall with 22 mm Thick Stucco:
  b. Fitted SDOF Model.
TABLE 1-2: Fitted Hysteretic Parameters for the SDOF Shear Element Model of a 2.4 m x 2.4 m Shear Wall with 22 mm Thick Stucco.

<table>
<thead>
<tr>
<th>$K_0$ (kN/mm)</th>
<th>$r_1$</th>
<th>$r_2$</th>
<th>$r_3$</th>
<th>$r_4$</th>
<th>$F_0$ (kN)</th>
<th>$F_1$ (kN)</th>
<th>$\Delta_u$ (mm)</th>
<th>$\alpha$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.00</td>
<td>0.058</td>
<td>-0.050</td>
<td>1.00</td>
<td>0.020</td>
<td>8.00</td>
<td>1.20</td>
<td>15.0</td>
<td>0.60</td>
<td>1.10</td>
</tr>
</tbody>
</table>

Even with experimental data from monotonic testing only it is possible to construct and calibrate SDOF shear elements to represent the expected cyclic shear wall behavior. For example, Fig. 1-6 shows the experimental results from the monotonic loading of a 2.4 m x 2.4 m shear wall sheathed on one-side only with 12 mm thick gypsum wallboard (GWB) (Gatto and Uang 2002). Under this experimental study this wall type was not subjected to cyclic loading. Nevertheless, the SDOF shear element model can be fitted to the monotonic test data to give the following model parameters: $F_0$, $K_0$, $r_1$, $r_2$, and $\Delta_u$ as presented in Table 1-3. The remaining model parameters $r_3$, $r_4$, $\alpha$ and $\beta$ listed in Table 1-3 were assigned values based on consideration of other tests conducted on the cyclic response of shear walls sheathed with GWB (McMullin and Merrick 2002).

The predicted cyclic response of the GWB shear wall by the SDOF shear element under the CUREE-Caltech testing protocol is shown in Fig 1-6. It can be seen that this model provides reasonably good agreement with the experimental monotonic curve when considered as an envelope to the cyclic response. Again, the information obtained here could be used to study the lateral stiffness and strength contributions of gypsum wallboard at the structural system level by using these calibrated shear wall spring elements in a building model.
### TABLE 1-3: Fitted and Assigned Hysteretic Parameters for the SDOF Shear Element Model of a 2.4 m x 2.4 m Shear Wall with 12 mm Thick Gypsum Wall Board

<table>
<thead>
<tr>
<th>$K_0$ (kN/mm)</th>
<th>$r_1$</th>
<th>$r_2$</th>
<th>$r_3$</th>
<th>$r_4$</th>
<th>$F_0$ (kN)</th>
<th>$F_I$ (kN)</th>
<th>$\Delta_u$ (mm)</th>
<th>$\alpha$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.60</td>
<td>0.029</td>
<td>-0.017</td>
<td>1.00</td>
<td>0.005</td>
<td>3.56</td>
<td>0.80</td>
<td>24.0</td>
<td>0.80</td>
<td>1.10</td>
</tr>
</tbody>
</table>

**Figure 1-6:** Experimental Monotonic Response and Predicted Cyclic Response of a 2.4 m x 2.4 m Shear Wall with 12 mm Thick Gypsum Wall Board.
1-3. GOVERNING EQUATIONS AND SOLUTION STRATEGIES

1-3.1. Dynamic Analysis

The governing equations of motion for the 3-D structure when subjected to an earthquake ground motion are given by:

$$\mathbf{M}\ddot{\mathbf{D}}(t) + \mathbf{C}\dot{\mathbf{D}}(t) + \mathbf{F}_{sw}(\mathbf{D}(t)) = -\mathbf{M}\ddot{x}_g(t)$$  \hspace{1cm} (9)

where \( \mathbf{M} \) is the global mass matrix; \( \mathbf{C} \) is the global viscous damping matrix; \( \mathbf{F}_{sw}(t) \) is the global restoring force vector generated by all of shear wall elements in the structure; \( \dot{\mathbf{D}}(t), \mathbf{D}(t) \) and \( \ddot{\mathbf{D}}(t) \) are, respectively, the global acceleration, velocity and displacement vectors relative to the ground; \( \ddot{x}_g(t) \) is the ground acceleration and \( \mathbf{r} \) is a vector coupling the ground motion input with the global degrees-of-freedom excited by that motion. The right hand side of (9) must be replaced by the more general expression

$$-\mathbf{M}\left[ \mathbf{r}_U\ddot{x}_g(t) + \mathbf{r}_V\ddot{x}_g(t) \right]$$

for the case of bi-directional ground accelerations \( \ddot{x}_{gU}(t) \) and \( \ddot{x}_{gV}(t) \) applied parallel to the x-axis and y-axis, respectively.

To advance the solution of (9) from time \( t \) to \( t+\Delta t \), the equations of motion are rewritten in incremental form:

$$\mathbf{M}\Delta\ddot{\mathbf{D}} + \mathbf{C}\Delta\dot{\mathbf{D}} + \Delta\mathbf{F}_{sw} = -\mathbf{M}\Delta\ddot{x}_g(t)$$  \hspace{1cm} (10)

with \( \Delta(-) = (-)_{t+\Delta t} - (-)_t \). Within each time step the structural response is assumed linear. As a consequence, the increment in the global restoring force is given by:

$$\Delta\mathbf{F}_{sw} = \mathbf{K}_T\Delta\mathbf{D}$$  \hspace{1cm} (11)

where \( \mathbf{K}_T \) is the global tangent stiffness matrix given by (4) and evaluated at time \( t \). Newmark’s Method (Chopra 2001) is used to integrate (10) over the time domain. With this integration scheme the system of incremental equations becomes:
\[ K_e \Delta D = \Delta P_e \]  \hspace{1cm} (12)

where the effective stiffness matrix \( K_e \) and effective load vector \( \Delta P_e \) are, respectively, given by:

\[
K_e = \frac{1}{\beta (\Delta t)^2} M + \frac{\tilde{\gamma}}{\beta \Delta t} C + K_T
\]  \hspace{1cm} (13)

\[
\Delta P_e = -Mr \Delta \ddot{x}_g + \left[ \frac{1}{2\beta} M + \left( \frac{\tilde{\gamma}}{2\beta} - 1 \right) \Delta t C \right] \ddot{D}_t + \left[ \frac{1}{\beta \Delta t} M + \frac{\tilde{\gamma}}{\beta} C \right] \dot{D}_t
\]  \hspace{1cm} (14)

with \( \tilde{\beta} \) and \( \tilde{\gamma} \) integration parameters. Assigning values of \( \tilde{\beta} = 1/4 \) and \( \tilde{\gamma} = 1/2 \) enforces the condition of constant average acceleration within a time step. The resulting integration scheme is unconditionally stable for linear systems.

As presented, this incremental solution strategy does not guarantee that equilibrium is achieved at the end of each time step. To minimize the accumulation of error over time, the relative acceleration at time \( t + \Delta t \) is determined by enforcing dynamic equilibrium of (9) at the end of each time step:

\[
\ddot{D}(t + \Delta t) = -M^{-1} \left[ Mr \ddot{x}_g (t + \Delta t) + CD(t + \Delta t) + F_{sw} (D(t + \Delta t)) \right]
\]  \hspace{1cm} (15)

In turn, this value of acceleration is used in the next time step in the evaluation of (14).

Energy balance calculations are performed at the end of each time step to monitor the accuracy of this solution strategy as well as to assess the energy absorption capacity of the shear wall elements of the structure. Pre-multiplying (9) by \( \ddot{D}^T \) and integrating up to time \( t \), yields the following energy balance equation in terms of relative global nodal displacements and velocities:

\[
E_K(t) + E_D(t) + E_A(t) = E_1(t)
\]  \hspace{1cm} (16)

where

\[
E_K(t) = \frac{1}{2} \ddot{D}^T(t)M\ddot{D}(t)
\]  \hspace{1cm} (17)
is the current relative kinetic energy;

\[ E_D(t) = \int \dot{D}^T C dD \]  

(18)

is the energy dissipated in the structure by viscous damping;

\[ E_A(t) = \int F_{su}^T dD \]  

(19)

is the energy absorbed by the shear wall elements; and

\[ E_1(t) = -\int (Mr\ddot{x}_g)^T dD \]  

(20)

is the relative seismic energy input. The process of integrating the equations of motion is generally aborted if (16) is not satisfied within a specified tolerance.

The global mass matrix \( M \) can be obtained through consideration of dynamic equilibrium and application of D’Alembert’s principle. With reference to Fig. 1-2, the global mass matrix for a single-story structure is given by:

\[
M = \begin{bmatrix}
m & 0 & -m y_c \\
0 & m & mx_c \\
-m y_c & mx_c & I_0 + m(x_c^2 + y_c^2)
\end{bmatrix}
\]  

(21)

where \( m \) is the mass of the diaphragm, \((x_c, y_c)\) are the coordinates of its center of mass and \( I_0 \) is the mass moment of inertia about the center of mass. As presented, (21) only represents the contributing mass from the diaphragm; mass associated with each shear wall can be represented by a discrete mass that includes rotatory inertia and added appropriately to (21).

The global viscous damping matrix \( C \) accounts for all supplemental energy dissipating mechanisms in the structure other than the hysteretic damping produced in the shear wall elements. In this formulation the viscous damping matrix is assumed to follow a Rayleigh damping model, such that:
\[ C = \alpha_M M + \beta_K K_0 \]  

(22)

where \( \alpha_M \) and \( \beta_K \) are, respectively, mass and stiffness proportional damping coefficients and \( K_0 \) is the global initial stiffness matrix for the structure. To obtain specified fractions of critical damping \( \xi_i \) and \( \xi_j \) in modes \( i \) and \( j \) with circular frequencies \( \omega_i \) and \( \omega_j \), respectively, \( \alpha_M \) and \( \beta_K \) are determined by:

\[
\alpha_M = \frac{2 \omega_i \omega_j (\omega_j \xi_i - \omega_i \xi_j)}{(\omega_j^2 - \omega_i^2)} \quad \beta_K = \frac{2 (\omega_j \xi_j - \omega_i \xi_i)}{(\omega_j^2 - \omega_i^2)}
\]

(23a,b)

As seen from (23), to evaluate \( \alpha_M \) and \( \beta_K \) the circular frequencies of the structures are first required. These are obtained from the solution of the free-vibration eigenvalue problem:

\[
(K_0 - \omega_i^2 M) F_i = 0 \quad i = 1, \ldots, \text{NDOF}
\]

(24)

where \( \omega_i \) and \( F_i \) are the \( i \)-th circular frequency and mode shape, respectively. The analysis performed by (24) also provides the modal frequencies and periods:

\[
f_i = \frac{\omega_i}{2\pi}; \quad T_i = \frac{2\pi}{\omega_i} \quad i = 1, \ldots, \text{NDOF}
\]

(25a,b)

Also, of importance from this modal analysis is the determination of the modal participation factor \( \Gamma_i \) associated with each mode:

\[
\Gamma_i = \frac{F_i^T M r}{F_i^T M F_i} \quad i = 1, \ldots, \text{NDOF}
\]

(26)

The modal participation factor \( \Gamma_i \) is an indicator of how much the \( i \)-th mode is excited by the acceleration of the earthquake.

To evaluate (22) and (24) given above, specification of the global initial stiffness matrix \( K_0 \) is required. This is determined by assigning the appropriate value of \( K_0 \), as given in (6), to each SDOF shear element in the structure.
1-3.2. Pushover Analysis

The governing equations of motion for the 3-D structure when subjected to a general dynamic force excitation are given by:

\[ M\ddot{D}(t) + C\dot{D}(t) + F_{sw}(D(t)) = P(t) \]  

(27)

where (27) is identical to (9) except that the right-hand side of the equation has been replaced by the global applied forcing function \( P(t) \). In order to perform a unidirectional quasi-static pushover analysis parallel to the x-axis or y-axis, using (27), \( P(t) \) is decomposed as follows:

\[ P(t) = p(t) \cdot F \]  

(28)

with \( p(t) \) representing the temporal variation in the slowly varying, monotonically increasing, applied pushover load, while \( F \) determines the distribution of nodal forces acting on the structure.

The time varying load \( p(t) \) takes the form:

\[ p(t) = \frac{1}{2} \left[ 1 - \cos \left( \frac{\pi t}{T_p} \right) \right] \cdot P_{\text{max}} \quad 0 \leq t \leq T_p \]  

(29)

where \( P_{\text{max}} \) represents the magnitude of the maximum lateral load applied to the structure and \( T_p \) is the duration of the applied loading. To minimize the contribution from inertia and damping effects in (27), the duration of loading \( T_p \) is set to one hundred times the fundamental period \( T_1 \) of the structure. Also, to ensure that the lateral load-carrying capacity of the structure is reached during the pushover analysis, using (28) and (29), \( P_{\text{max}} \) is set equal to:

\[ P_{\text{max}} = (0.05H_T) \cdot \frac{P_1}{\Delta_1} \]  

(30)

where \( H_T \) is the total height of the structure, \( P_1 \) is a lateral load of unit magnitude and \( \Delta_1 \) is the corresponding top of roof displacement obtained from a linear quasi-static analysis. In essence,
using (30) assumes that the pushover load will be less than the load required to produce a 5% drift in the structure under a linear elastic response.

The distribution of the lateral loading applied at the global degrees-of-freedom is determined through $F$. First off, $F$ represents nodal loads equivalent to pushover loads applied at the center of mass of each floor of the structure. Three options on the distribution of loading over the height of the structure are considered in this analysis: a uniform distribution, an inverted triangular distribution and a modal adaptive distribution. Each of these lateral loads distributions is illustrated in Fig. 1-6.

![Figure 1-6: Distributions of Lateral Loads for Static Pushover Analysis](image-url)

**Figure 1-6:** Distributions of Lateral Loads for Static Pushover Analysis:
- a. Uniform Distribution
- b. Inverted Triangular Distribution
- c. Modal Adaptive Distribution
The uniform load distribution assigns to each floor level, over the height of the structure, a constant lateral force. For this case, the force \( F_i \) applied at the center of mass at each story level is simply given by:

\[
F_i = \frac{V_b}{N_s} \quad i = 1, \ldots, N_s
\]

(31)

where \( N_s \) is the number of stories in the structure and \( V_b \) is the shear force developed at the base of the structure. An assumed uniform distribution amplifies the seismic demand imposed on the lower stories of the structure.

The inverted triangular load distribution assigns lateral forces at each floor level according to the relationship:

\[
F_i = \left( \frac{W_i h_i}{\sum_{j=1}^{N_s} W_j h_j} \right) \cdot V_b \quad i = 1, \ldots, N_s
\]

(32)

where \( W_i \) and \( h_i \) are, respectively, the weight and elevation (relative to the foundation) of the i-th floor level. This distribution assumes that the structure is subjected to a linear variation in lateral acceleration over the height of the structure and is associated with a first mode response.

The lateral distribution applied during the pushover analysis using the above two approaches does not account for redistribution of loading under the evolving inelastic response of the structure up to collapse. To provide a better representation of this, the lateral loads can be assigned using the following modal adaptive distribution (Valles et al. 1996):
where $\Phi_{ij}$ is the value of the $j$-th mode shape at the $i$-th floor level, $\Gamma_j$ is the modal participation factor for the $j$-th mode. The numerator and denominator of (33) utilizes a square root of the sum of the square (SRSS) weighting applied over the first $N_m$ significant mode shapes, each scaled by their respective modal participation factor. This method is adaptive in that the lateral load distribution is continually updated after a specified number of time steps following the determination of the current mode shapes and the modal participation factors from an eigenvalue analysis using (24), but with $K_0$ replaced by $K_T$.

It is to be noted that a static pushover analysis as a procedure for evaluating the seismic performance of a structure has no rigorous theoretical foundation (Krawinkler and Seneviratna 1998). In particular, specification of an appropriate lateral load distribution is a weak point in this methodology. Nevertheless, the general expectation is that such an analysis will provide adequate information on the seismic demands imposed on the overall structural system and its components. At least being able to utilize the three different distribution presented above can help determine the response sensitivity of a given structure to applied lateral loadings.

### 1-4. CONCLUDING REMARKS

A recent invitational workshop on seismic testing, analysis and design of woodframe construction (Seible et al 1999), attend by design practitioner and researchers, concluded that there was an urgent need to develop simple dedicated structural analysis tools for woodframe

$$F_i = \left( \frac{W_i \left[ \sum_{j=1}^{N_m} (\Phi_{ij} \Gamma_j)^2 \right]^{1/2}}{\sum_{k=1}^{N_m} W_k \left[ \sum_{j=1}^{N_m} (\Phi_{kj} \Gamma_j)^2 \right]^{1/2}} \right) V_b \quad i, k = 1, \ldots, N_i; \quad j = 1, \ldots, N_{DOF}$$ (33)
building systems. Addressing this need, a simple, versatile numerical model to predict the
dynamic characteristics, quasi-static pushover and seismic response of light-frame wood buildings
has been presented herein. In this model the lateral load-resisting system of the wood building is
assumed to be composed of only two primary structural components: rigid horizontal diaphragms
and nonlinear shear wall elements. The actual three-dimensional building is degenerated into a
two-dimensional planar model using zero-height SDOF shear spring elements to interconnect the
diaphragms and to tie the structure to a rigid foundation. These SDOF shear spring elements are
capable of displaying pinched, strength and stiffness degrading hysteretic behavior, typifying the
cyclic response of wood shear walls. It has been shown that calibration of the model parameters
for the SDOF shear elements can be performed using the companion computer program
CASHEW - Cyclic Analysis of SHear Walls or by fitting the model to experimental data from full-
scale cyclic shear wall tests. With this simple modeling approach the response of the building is
defined in terms of only three-degrees-of-freedom per story. This results in a computationally
efficient structural analysis tool. The model formulation and solution strategies presented herein
have been incorporated into the computer program SAWS: Seismic Analysis of Woodframe
Structures. In Part 2 of this report the predictive capabilities of this program are compared with
recent shake table tests performed on a full-scale two-story residential house.
1-5. REFERENCES FOR PART 1


PART 2

NONLINEAR ANALYSIS OF WOODFRAME BUILDINGS – MODEL IMPLEMENTATION & VERIFICATION
2-1. INTRODUCTION

A very limited number of tests have been conducted in North America on full-scale light-frame residential wood buildings over the past 30 years (Yokel et al. 1973; Yancy et al. 1973; Tuomi et al. 1974; Phillips 1990; and Fischer et al. 2001). Of these experimental studies, only one (Fischer et al. 2001) investigated the dynamic response of a woodframe house through shake table testing. The remaining studies were limited to evaluating building response under monotonic and/or cyclic loading. As noted in Part 1 of this report, over nearly the same time period an equally limited number of 3-D structural analysis models for woodframe buildings have been developed (Gupta and Kuo 1987; Schmidt and Moody 1989; Yoon and Gupta 1991; Ge 1991; Kasal et al. 1994; Tarabai and Itani 1997). Of these, only one model is capable of performing a dynamic analysis (Tarabai and Itani 1997). It is therefore not surprising that a recent summary of research work conducted on the structural performance of woodframe housing identified full-scale house tests and the development of 3-D numerical models for woodframe houses as outstanding research needs (Yancey et al. 1998). The urgency for these same research activities was also echoed at a recent invitational workshop on seismic testing, analysis and design of woodframe construction (Seible et al. 1999).

These research needs are currently being addressed in part by the CUREE-Caltech Woodframe Project, which is a combined research and implementation effort to improve the seismic performance of woodframe buildings. One of the primary elements of the CUREE-Caltech Woodframe Project is testing and analysis. The scope of activities under this element of the project broadly covers the experimental testing of large-scale woodframe structures and structural components along with the associated analysis at the structure and component levels. A
key research activity within this element is Task 1.1.1: Shake Table Testing of a Two-Story Woodframe House (Fischer et al. 2001). Experimental results from this task will be used to evaluate the predictive capability of the 3-D structural analysis model SAWS - Seismic Analysis of Wood Structures presented previously in Part 1 of this report.

2-2. CUREE-CALTECH WOODFRAME PROJECT TEST STRUCTURE

2-2.1. Description of the Task 1.1.1 Test Structure

The test structure investigated under Task 1.1.1 (Shake Table Tests of a Two-Story Woodframe House) of the CUREE-Caltech Woodframe Project represents a simplified full-scale two-story single-family house incorporating several characteristics of recent California residential construction (Fischer et al. 2001). The overall testing program covered by Task 1.1.1 involved 10 different construction phases of the test structure. Only the fully engineered (Phase 9 and 10) test structures, as described below, will be used in the validation procedure of the SAWS model.

Figure 2-1 shows plan views of the first and second stories of the test structure. Elevation views are shown in Fig. 2-2. The major structural components of the test structure are identified in these two figures. For Phase 9 and 10, the test structure shares the same building layout. The Phase 9 test structure consists of only the bare woodframing as shown in Fig. 2-3, while the Phase 10 test structure includes all interior and exterior wall finishes (gypsum wallboard and stucco) along with complete window and door details, as shown in Fig. 2-4.

The construction of the test structure was at full scale. The plan dimensions, however, are smaller than would be of a typical residence due to limitations of the shake table. All tests conducted on the shake table facility were unidirectional along the short dimension of the
structure (north-south direction). The lateral load-resisting system of the test structure parallel to the shaking direction (east and west wall elevations) consisted of exterior shear walls. As seen in Figs. 2-3 and 2-4, a large garage door opening was included in the east wall. This asymmetry was incorporated into the structure to produce a torsional eccentricity in the lateral load resisting system. The exterior shear walls also supported the gravity loads together with a first-floor interior bearing wall and a glued-laminated beam at the floor level. All shear walls were sheathed with 9.5 mm thick oriented strand board (OSB) panels that were fastened to the framing with 8-penny box gun nails. The second floor exterior walls were tied to the first floor walls by steel straps and the first floor walls were tied-down to the foundation by steel connector hardware at all openings and anchor bolts distributed around the foundation, as shown in Fig. 2-2.

![Plan View of the Test Structure Showing Major Structural Components.](image)

**Figure 2-1:** Plan View of the Test Structure Showing Major Structural Components.
A finished tile roof was installed for both Phase 9 and 10 testing. To have a constant mass for the structure across the two test phases, concrete blocks were strategically added to the Phase 9 test structure.
For Phase 10, the wall finish materials consisted of interior gypsum wallboard (GWB) and exterior stucco. The 12 mm thick gypsum wallboard panels were installed on all interior wall and ceiling surfaces. All surfaces were taped, mudded and painted. The panels were fastened with 32...
mm long drywall screws. The exterior stucco finish was applied in three coats for a total thickness of 22 mm. Cylinder tests on the stucco indicated an average compressive strength of 9 MPa at the time of testing. The stucco was attached to the woodframing by 17-gage galvanized steel wire lath fastened to the OSB sheathing and vertical studs by 20 mm long staples.

The design of the Phase 9 and 10 test structures was based on the engineering provisions of the 1994 edition of the Uniform Building Code (ICBO 1994) for a seismic zone 4 and common design practices in California. The design assumes a force-reduction factor $R_w$ of 8, for a lateral load-resisting system consisting light-frame wood shear walls. The seismic weight of the structure was 110 kN and the period of vibration estimated by the code was 0.18 s. The resulting design base shear in the shaking direction was 15 kN. The ultimate base shear capacity of the test structure was estimated to be 75 kN, according to the FEMA 273 guidelines (ATC 1997).

A complete description of the Phase 9 and 10 test structures is presented in CUREE Publication No. W-06: *Shake Table Tests of a Two-story Woodframe House*, (Fischer et al. 2001).

### 2-2.2. Description of the Task 1.1.1 Phase 9 & 10 Testing

During both Phase 9 and 10 of Task 1.1.1, the test structure was subjected to an extensive series of dynamic tests, which included steady state, white noise and seismic ground motions. The white noise was used to determine frequencies and mode shapes of the structure under low amplitude vibrations. The steady state input was used to determine damping characteristics. The seismic ground motions were used to determine the inelastic response of the structure under increasing intensities of shaking. All of this testing was performed on the 3 m x 4.9 m uniaxial earthquake simulation system at the University of California, San Diego.
The test structure was anchored to the shake table by a specially fitted stiff, steel base that was bolted to the top of the shake table. This base incorporated outrigger arms to accommodate the footprint of the test structure. Threaded steel studs, welded to the top flange of the steel base, were used as anchor bolts for the sill plates of the test structure. Also, a 38 mm thick layer of grout was used between the steel base and the sill plate to simulate a concrete foundation for the test structure.

For both Phase 9 and 10, five levels of seismic tests were performed on the test structure. The input ground motion for seismic test levels 1 to 4 represents a scaling of the 1994 Northridge Earthquake recorded at Canoga Park, as shown in Table 2-1. The input ground motion for seismic test level 5 represents the unscaled 1994 Northridge Earthquake recorded at Rinaldi. Seismic test level 4 represents a ground motion with a hazard level of 10% probability of exceedance in 50 years. Seismic test level 5 represents a ground motion with a hazard level of 2% probability of exceedance in 50 years. Figure 2-5 presents the acceleration time-histories along with the absolute acceleration response spectra at 5% damping for these two seismic records. As part of the test program, one seismic test was repeated once the maximum transient inter-story wall drift exceeded 0.5% of its height. For Phase 9 this occurred at level 3, while for Phase 10 it occurred at level 4. The designations for these tests are 9.S.3R and 10.S.4R, respectively.

Over Phase 9 the test structure was subjected, in turn, to each one of these ground motions without any repair or modification to the structure. Between Phase 9 and 10 the test structure was extensively repaired to bring it back, as much as possible, to its initial undamaged state. Then over Phase 10, testing continued without repairs being made to the test structure.
Table 2-1: Ground Motions for Seismic Tests

<table>
<thead>
<tr>
<th>Seismic Test Level</th>
<th>Test Designation</th>
<th>Ground Motion</th>
<th>Amplitude Scaling Factor</th>
<th>Peak Ground Acceleration (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>9.S.1 10.S.1</td>
<td>1994 Northridge</td>
<td>0.12</td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Canoga Park</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>9.S.2 10.S.2</td>
<td>1994 Northridge</td>
<td>0.53</td>
<td>0.22</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Canoga Park</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 &amp; 3R</td>
<td>9.S.3 &amp; 9.S.3R</td>
<td>1994 Northridge</td>
<td>0.86</td>
<td>0.36</td>
</tr>
<tr>
<td></td>
<td>10.S.3</td>
<td>Canoga Park</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4 &amp; 4R</td>
<td>9.S.4 10.S.4 &amp; 10.S.4R</td>
<td>1994 Northridge</td>
<td>1.2</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>10.S.3</td>
<td>Canoga Park</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>9.S.5 10.S.5</td>
<td>1994 Northridge</td>
<td>1.00</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Rinaldi Park</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The test structure was monitored with nearly three hundred digital instruments to measure forces, displacements and accelerations in the structure during the shake table tests. For the evaluation of the SAWS model response quantities of interest will be limited to relative displacement and absolute acceleration time-histories at the roof level. Displacement and acceleration data acquisition sensors were placed at roof level along the east wall, center-line and west wall of the structure, as shown in Fig. 2-6.

For a complete presentation of all of the testing performed under Task 1.1.1, one is referred to CUREE Publication No. W-06: *Shake Table Tests of a Two-story Woodframe House*, (Fischer et al. 2001).
Figure 2-5: Earthquake Ground Motions Used in the Seismic Tests and their Absolute Acceleration Response Spectra for 5% Damping.
2-3. **SAWS MODEL IMPLEMENTATION**

As presented in Part 1 of this report the SAWS computer program was formulated to predict the dynamic characteristics, quasi-static pushover and seismic response of light-frame wood buildings. With this model, the actual three-dimensional building is degenerated into a two-dimensional planar model composed of zero-height shear wall spring elements connecting the floor and roof diaphragms together or to the foundation. All diaphragms in the building model are assumed to have infinite in-plane stiffness. Using this simple modeling approach, the response of the building is defined in terms of only 3-DOF per floor. Each shear wall spring element is calibrated to reflect the pinched, strength and stiffness degrading hysteretic behavior of the shear wall it is modeling in the structure. Requisite test data is limited to that necessary to characterize...
the behavior of the shear walls. Implementation of this model for the analysis of the CUREE-Caltech Woodframe Project Task 1.1.1 test structure from Phase 9 and 10 testing follows.

2-3.1. **SAWS Model of the Phase 9 Test Structure**

The Task 1.1.1 - Phase 9 test structure consists of only the bare woodframing, as shown in Fig. 2-3. The SAWS model for this structure, which is schematically shown in Fig. 2-7, is composed of eight zero-height shear wall spring elements and two rigid diaphragms; one for the second floor and one at the roof level. The force-deformation response of each shear wall spring element requires the specification of ten hysteretic parameters, as discussed in Section 1-2.4. These parameters can be determined using the companion analysis program CASHEW - Cyclic Analysis of SHEar Walls (Folz and Filiatrault 2000). CASHEW determines the cyclic response of wood shear walls under the CUREE-Caltech Woodframe Project Testing Protocol (Krawinkler et al. 2000). Data input to CASHEW requires specification of the wall geometry, shear stiffness of the sheathing panels and the hysteretic properties of the sheathing-to-framing connectors. All of this data was determined under Task 1.1.1 (Fischer et al. 2001). As determined by the CASHEW program, the calibrated hysteretic parameters required to describe the cyclic response of the eight Phase 9 shear walls are given Table 2-2. Also shown in Fig. 2-8 are the CASHEW model predictions of the cyclic response of the first and second level east shear walls (designated as $S_{X1}$ and $S_{X3}$ in Fig. 2-7 and Table 2-2) under the CUREE-Caltech Testing Protocol.
Figure 2-7: SAWS Model of the Task 1.1.1 - Phase 9 Shake Table Test Structure.
Table 2-2: Hysteretic Parameters for the Shear Wall Spring Elements in Task 1.1.1 - Phase 9 Test Structure.

<table>
<thead>
<tr>
<th>Spring Element</th>
<th>$K_0$ (kN/mm)</th>
<th>$r_1$</th>
<th>$r_2$</th>
<th>$r_3$</th>
<th>$r_4$</th>
<th>$F_0$ (kN)</th>
<th>$F_I$ (kN)</th>
<th>$Δ_u$ (mm)</th>
<th>$α$</th>
<th>$β$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_{X1}$ Level 1 East Wall</td>
<td>2.93</td>
<td>0.083</td>
<td>-0.088</td>
<td>1.00</td>
<td>0.03</td>
<td>36.6</td>
<td>8.36</td>
<td>87.3</td>
<td>0.79</td>
<td>1.07</td>
</tr>
<tr>
<td>$S_{X2}$ Level 1 West Wall</td>
<td>3.89</td>
<td>0.064</td>
<td>-0.056</td>
<td>1.07</td>
<td>0.03</td>
<td>36.7</td>
<td>8.82</td>
<td>57.8</td>
<td>0.87</td>
<td>1.11</td>
</tr>
<tr>
<td>$S_{Y1}$ Level 1 South Wall</td>
<td>5.69</td>
<td>0.065</td>
<td>-0.074</td>
<td>1.10</td>
<td>0.03</td>
<td>48.4</td>
<td>10.8</td>
<td>60.6</td>
<td>0.81</td>
<td>1.09</td>
</tr>
<tr>
<td>$S_{Y2}$ Level 1 North Wall</td>
<td>5.69</td>
<td>0.065</td>
<td>-0.074</td>
<td>1.10</td>
<td>0.03</td>
<td>48.4</td>
<td>10.8</td>
<td>60.6</td>
<td>0.81</td>
<td>1.09</td>
</tr>
<tr>
<td>$S_{X3}$ Level 2 East Wall</td>
<td>2.10</td>
<td>0.069</td>
<td>-0.038</td>
<td>1.16</td>
<td>0.02</td>
<td>19.6</td>
<td>4.76</td>
<td>76.8</td>
<td>0.77</td>
<td>1.10</td>
</tr>
<tr>
<td>$S_{X4}$ Level 2 West Wall</td>
<td>2.10</td>
<td>0.069</td>
<td>-0.038</td>
<td>1.16</td>
<td>0.02</td>
<td>19.6</td>
<td>4.76</td>
<td>76.8</td>
<td>0.77</td>
<td>1.10</td>
</tr>
<tr>
<td>$S_{Y3}$ Level 2 South Wall</td>
<td>3.35</td>
<td>0.054</td>
<td>-0.060</td>
<td>1.10</td>
<td>0.03</td>
<td>35.3</td>
<td>12.9</td>
<td>73.9</td>
<td>0.84</td>
<td>1.09</td>
</tr>
<tr>
<td>$S_{Y4}$ Level 2 North Wall</td>
<td>3.35</td>
<td>0.054</td>
<td>-0.060</td>
<td>1.10</td>
<td>0.03</td>
<td>35.3</td>
<td>12.9</td>
<td>73.9</td>
<td>0.84</td>
<td>1.09</td>
</tr>
</tbody>
</table>
Figure 2-8: CASHEW Predictions of the Cyclic Response of the 1st and 2nd Level East Shear Walls of the Task 1.1.1 – Phase 9 Test Structure.

2-3.2. **SAWS Model of the Phase 10 Test Structure**

The Task 1.1.1 - Phase 10 test structure includes both exterior (stucco) and interior (gypsum wall board) finishes on the engineered woodframing, as shown in Fig. 2-4. The SAWS model for this structure, which is schematically represented in Fig. 2-9, is composed of twenty-seven zero-height shear spring elements to represent the eight exterior walls covered with stucco, the eight OSB sheathed shear walls and the eleven walls covered with gypsum wallboard. The SAWS model of the test structure also includes two rigid diaphragms; one for the second floor and one at the roof level.

For the Phase 10 test structure, the hysteretic parameters required to characterize the shear spring elements are obtained using the CASHEW program for the walls with OSB sheathing and from test data for the walls with GWB and with stucco. The required hysteretic parameters to
model the Phase 10 OSB sheathed shear walls are identical to those determined under Phase 9. The hysteretic parameters for walls with stucco are obtained by adapting available test data (Pardoen 2000) to the wall configurations in the test structure. The calibrated hysteretic parameters for a 2.4 m x 2.4 m wall with 22 mm thick stucco have been determined previously (see Table 1-2). For the walls in the Phase 10 test structure, these values are adjusted for the length of the walls and for the presence of door and window openings. Only full wall piers are considered in this calibration. Similarly, the hysteretic parameters for walls with GWB are obtained by adapting available test data (Gatto and Uang 2002) to the wall configurations in the test structure. The calibrated hysteretic parameters for a 2.4 m x 2.4 m wall with 12 mm thick gypsum wallboard have also been determined previously (see Table 1-3). In a similar fashion these hysteretic parameter values are adjusted to reflect the configuration of these walls in the test structure. In addition, it is to be noted that all interior partition walls have gypsum wallboard on two sides. For this case it is assumed that the strength and stiffness of these walls are double that of walls with GWB on only one side. The resulting hysteretic parameters for all of the twenty-seven walls in the Phase 10 test structure are specified in Table 2-3.
Figure 2-9: SAWS Model of the Task 1.1.1 - Phase 10 Shake Table Test Structure.
Table 2-3: Hysteretic Parameters for Shear Wall Spring Elements in Task 1.1.1 - Phase 10 Test Structure.

<table>
<thead>
<tr>
<th>Spring Element</th>
<th>Wall Type &amp; Location</th>
<th>$K_0$ (kN/mm)</th>
<th>$r_1$</th>
<th>$r_2$</th>
<th>$r_3$</th>
<th>$r_4$</th>
<th>$F_0$ (kN)</th>
<th>$F_I$ (kN)</th>
<th>$\Delta_0$ (mm)</th>
<th>$\alpha$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S_{X1}</td>
<td>Stucco Level 1 - East Wall</td>
<td>3.75</td>
<td>0.058</td>
<td>-</td>
<td>0.050</td>
<td>1.00</td>
<td>0.03</td>
<td>30.0</td>
<td>6.00</td>
<td>11.3</td>
<td>0.60</td>
</tr>
<tr>
<td>S_{X2}</td>
<td>OSB Level 1 - East Wall</td>
<td>2.93</td>
<td>0.083</td>
<td>-</td>
<td>0.088</td>
<td>1.00</td>
<td>0.03</td>
<td>36.6</td>
<td>8.36</td>
<td>87.3</td>
<td>0.79</td>
</tr>
<tr>
<td>S_{X3}</td>
<td>GWB (1 Side) Level 1 - East Wall</td>
<td>1.95</td>
<td>0.029</td>
<td>-</td>
<td>0.017</td>
<td>1.00</td>
<td>0.005</td>
<td>2.67</td>
<td>0.60</td>
<td>18.4</td>
<td>0.80</td>
</tr>
<tr>
<td>S_{X4}</td>
<td>GWB (2 Sides) Level 1 - Partition Wall</td>
<td>3.90</td>
<td>0.029</td>
<td>-</td>
<td>0.017</td>
<td>1.00</td>
<td>0.005</td>
<td>5.34</td>
<td>1.20</td>
<td>36.8</td>
<td>0.80</td>
</tr>
<tr>
<td>S_{X5}</td>
<td>Stucco Level 1 - West Wall</td>
<td>8.13</td>
<td>0.058</td>
<td>-</td>
<td>0.050</td>
<td>1.00</td>
<td>0.03</td>
<td>13.0</td>
<td>1.95</td>
<td>24.4</td>
<td>0.60</td>
</tr>
<tr>
<td>S_{X6}</td>
<td>OSB Level 1 - West Wall</td>
<td>3.89</td>
<td>0.064</td>
<td>-</td>
<td>0.056</td>
<td>1.07</td>
<td>0.03</td>
<td>36.7</td>
<td>8.82</td>
<td>57.8</td>
<td>0.87</td>
</tr>
<tr>
<td>S_{X7}</td>
<td>GWB (1 Side) Level 1 - West Wall</td>
<td>4.23</td>
<td>0.029</td>
<td>-</td>
<td>0.017</td>
<td>1.00</td>
<td>0.005</td>
<td>5.79</td>
<td>1.30</td>
<td>39.0</td>
<td>0.80</td>
</tr>
<tr>
<td>S_{Y1} &amp; S_{Y4}</td>
<td>Stucco Level 1 – South &amp; North Walls</td>
<td>8.75</td>
<td>0.058</td>
<td>-</td>
<td>0.050</td>
<td>1.00</td>
<td>0.03</td>
<td>14.0</td>
<td>2.10</td>
<td>26.3</td>
<td>0.60</td>
</tr>
<tr>
<td>S_{Y2} &amp; S_{Y5}</td>
<td>OSB Level 1 - South Wall</td>
<td>5.69</td>
<td>0.065</td>
<td>-</td>
<td>0.074</td>
<td>1.10</td>
<td>0.03</td>
<td>48.4</td>
<td>10.8</td>
<td>60.6</td>
<td>0.81</td>
</tr>
<tr>
<td>S_{Y3} &amp; S_{Y6}</td>
<td>GWB (1 Side) Level 1 - South &amp; North Walls</td>
<td>4.55</td>
<td>0.029</td>
<td>-</td>
<td>0.017</td>
<td>1.00</td>
<td>0.005</td>
<td>6.23</td>
<td>1.40</td>
<td>42.0</td>
<td>0.80</td>
</tr>
</tbody>
</table>
Table 2-3: Hysteretic Parameters for Shear Wall Spring Elements in Task 1.1.1 - Phase 10 Test Structure (cont.)

<table>
<thead>
<tr>
<th>Spring Element</th>
<th>Wall Type &amp; Location</th>
<th>$K_0$ (kN/mm)</th>
<th>$r_1$</th>
<th>$r_2$</th>
<th>$r_3$</th>
<th>$r_4$</th>
<th>$F_0$ (kN)</th>
<th>$F_I$ (kN)</th>
<th>$\Delta_u$ (mm)</th>
<th>$\alpha$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>SX8 &amp; SX13</td>
<td>Stucco Level 2 - East &amp; West Walls</td>
<td>4.38</td>
<td>0.058</td>
<td>-</td>
<td>1.00</td>
<td>0.03</td>
<td>7.00</td>
<td>1.05</td>
<td>13.1</td>
<td>0.60</td>
<td>1.10</td>
</tr>
<tr>
<td>SX9 &amp; SX14</td>
<td>OSB Level 2 - East Wall</td>
<td>2.10</td>
<td>0.069</td>
<td>-</td>
<td>1.16</td>
<td>0.02</td>
<td>19.6</td>
<td>4.76</td>
<td>76.8</td>
<td>0.77</td>
<td>1.10</td>
</tr>
<tr>
<td>SX10 &amp; SX15</td>
<td>GWB (1 Side) Level 2 - East &amp; West Walls</td>
<td>2.28</td>
<td>0.029</td>
<td>-</td>
<td>1.00</td>
<td>0.005</td>
<td>3.12</td>
<td>0.70</td>
<td>21.0</td>
<td>0.60</td>
<td>1.10</td>
</tr>
<tr>
<td>SX11 &amp; SX12</td>
<td>GWB (2 Sides) Level 2 - Partition Walls</td>
<td>8.65</td>
<td>0.029</td>
<td>-</td>
<td>1.00</td>
<td>0.005</td>
<td>11.8</td>
<td>2.67</td>
<td>79.85</td>
<td>0.80</td>
<td>1.10</td>
</tr>
<tr>
<td>SY7 &amp; SY10</td>
<td>Stucco Level 2 – South &amp; North Walls</td>
<td>7.50</td>
<td>0.058</td>
<td>-</td>
<td>1.00</td>
<td>0.03</td>
<td>12.0</td>
<td>1.80</td>
<td>22.5</td>
<td>0.60</td>
<td>1.10</td>
</tr>
<tr>
<td>SY8 &amp; SY11</td>
<td>OSB Level 2 - South Wall</td>
<td>3.35</td>
<td>0.054</td>
<td>-</td>
<td>1.10</td>
<td>0.03</td>
<td>35.3</td>
<td>12.9</td>
<td>73.9</td>
<td>0.84</td>
<td>1.09</td>
</tr>
<tr>
<td>SY9 &amp; SY12</td>
<td>GWB (1 Side) Level 2 - South &amp; North Walls</td>
<td>3.90</td>
<td>0.029</td>
<td>-</td>
<td>1.00</td>
<td>0.005</td>
<td>5.34</td>
<td>1.20</td>
<td>36.0</td>
<td>0.80</td>
<td>1.10</td>
</tr>
</tbody>
</table>
2-4. **SAWS MODEL VERIFICATION**

As presented in Part 1 of this report, the SAWS computer program was formulated to predict the dynamic characteristics, quasi-static pushover and seismic response of woodframe buildings. In this section the predictive capabilities of the SAWS program will be compare with the experimental results obtained from the CUREE-Caltech Woodframe Project Task 1.1.1 shake table test structure under Phase 9 and 10 testing.

2-4.1. *SAWS Model Prediction of the Phase 9 Test Structure.*

The SAWS model of the Task 1.1.1 - Phase 9 shake table test structure is shown in Fig. 2-7 and the assigned properties for each shear wall spring element are given in Table 2-2. Using this model the predicted fundamental frequency of the structure is 3.28 Hz. This prediction is based on assigned seismic weights of 62 kN to the second floor diaphragm and 48 kN to the roof diaphragm. At the start of Phase 9 testing, the experimentally measured fundamental frequency was 3.96 Hz (Fischer et al. 2001). The SAWS model under predicts the experimental result by 17%. Potential sources for this difference are the inherent simplicity of the SAWS model and the CASHEW predictions of the initial stiffness of each shear wall element.

Figures 2-10 through 2-13 present the SAWS model predictions of the relative displacement and absolute acceleration time-histories along the centerline of the test structure at roof level for Phase 9 – level 4 and 5 testing along with the corresponding experimental results. These relative displacement and absolute acceleration measurements correspond to sensors C16 and D16, as shown in Fig. 2-6. The SAWS model predictions are based on an assigned equivalent viscous damping of 1% of critical in the first and second modes of vibration. It is assumed that
under the strong motion portions of shaking of levels 4 and 5 that damping in the structure can largely be accounted for through the hysteretic response of the shear wall spring elements. As a consequence, the viscous damping is set to a low value. This modeling approach is consistent with what has been suggested by other research work (Foliente 1995; Folz and Filiatrault 2001). Experimentally, the equivalent viscous damping for the test structure was calculated to be 4.2% of critical under small amplitude excitation.

With respect to maximum response values, over level 4, the maximum relative displacement and absolute acceleration at sensor location C16 and D16 estimated by the SAWS model under predict the experimental values by 17.6% and 18.2%, respectively. Over level 5, the maximum relative displacement and absolute acceleration at sensor location C16 and D16 estimated by the SAWS model under predict the experimental values by 6.3% and 9.6%, respectively. Table 2-4 provides a complete listing of the maximum response values predicted by the SAWS model and obtained experimentally for all of the sensor locations shown in Fig. 2-6. Comparing the maximum relative displacement results given in Table 2-4, the differences in the numerical predictions and the experimental values can, in part, be attributed to the SAWS model not properly capturing the torsional response of the test structure and also the model’s inability to account for the in-plane deformation in the roof diaphragm. Over test levels 4 and 5, the maximum torsional response resulted in a difference in relative displacement between the east and west walls of 5.2 mm and 11.7 mm, respectively. Also for levels 4 and 5, the maximum roof diaphragm deformations were 5.8 mm and 7.1 mm, respectively.
Figure 2-10: Phase 9 – Level 4 Relative Displacement Time Histories at Sensor C16:
  a) SAWS Prediction
  b) Experimental Result.
Figure 2-11: Phase 9 – Level 4 Absolute Acceleration Time Histories at Sensor D16:
a) SAWS Prediction
b) Experimental Result.
Figure 2-12: Phase 9 – Level 5 Relative Displacement Time Histories at Sensor D16:
   a) SAWS Prediction
   b) Experimental Result.
Figure 2-13: Phase 9 – Level 5 Absolute Acceleration Time Histories at Sensor D16:
   a) SAWS Prediction
   b) Experimental Result.
Table 2-4: Hysteretic Parameters for the Shear Wall Spring Elements in Task 1.1.1 - Phase 9 Test Structure.

<table>
<thead>
<tr>
<th>Sensor Locations</th>
<th>Level 4 – Maximum Relative Displacement (mm)</th>
<th>Level 4 – Maximum Absolute Acceleration (g)</th>
<th>Level 5 – Maximum Relative Displacement (mm)</th>
<th>Level 5 – Maximum Absolute Acceleration (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C14</td>
<td>57.3</td>
<td>0.99</td>
<td>102.8</td>
<td>1.35</td>
</tr>
<tr>
<td>C16</td>
<td>57.4</td>
<td>0.99</td>
<td>102.7</td>
<td>1.23</td>
</tr>
<tr>
<td>C18</td>
<td>57.5</td>
<td>1.05</td>
<td>102.6</td>
<td>1.18</td>
</tr>
<tr>
<td>% Error</td>
<td>-6.5</td>
<td>-14.7</td>
<td>6.3</td>
<td>6.3</td>
</tr>
<tr>
<td></td>
<td>-17.6</td>
<td>-19.6</td>
<td>-6.3</td>
<td>-9.6</td>
</tr>
<tr>
<td></td>
<td>-13.5</td>
<td>-7.01</td>
<td>5.4</td>
<td>-15.1</td>
</tr>
</tbody>
</table>

These dynamic time history results were produced by the SAWS program using a time integration step of 0.001 sec. In order to properly capture the evolution of damage to the structure over the Phase 9 testing regime the SAWS program was run using the train of shake table acceleration input records from level 3 to 5. It was noted from the experimental results that very little damage occurred to the Phase 9 structure under level 1 and 2 testing. The numerical
A computer program for seismic analysis of woodframe structures was applied with a specified tolerance of 5% on the error in the energy balance applied to the equations of motion. The computational time required to perform a given level of analysis on the Phase 9 test structure using the SAWS program on a modern desktop computer was less than a minute. This fast computational turn-around time is attributable to the fact the SAWS program models the two-story shake table test structure using only six degrees-of-freedom.

A static pushover test was not performed on the shake table structure. However, from the seismic test data from Phase 9 a capacity spectrum was generated as shown in Fig. 2-14. The capacity spectrum is a plot of the maximum base shear (obtained as the sum of the floor and roof inertia forces) versus the corresponding peak roof relative displacement (measured at sensor location C16) for each test level of the testing phase (Fischer et al. 2001). With the capacity spectrum, the computed base shear represents the force induced in the foundation of the test structure including the nonlinear restoring force and viscous damping force. In a static pushover only the nonlinear restoring force is activated since the resulting velocity of the structure is negligible during such a test. Thus, it is to be expected that the maximum base shear predicted by a static pushover analysis would be less than that estimated by a capacity spectrum. This is the case with the SAWS model static pushover results. Figure 2-14 shows the SAWS model predictions from three static pushover analyses, corresponding to lateral load distributions that are uniform, triangular and modal adaptive. These three results are in relatively close agreement with each other and predict a maximum base shear for the Phase 9 test structure of approximately 100 kN. For comparative purposes, design estimates of maximum base shear are also shown in Fig. 2-14.
2-4.2. **SAWS Model Prediction of the Phase 10 Test Structure.**

The Saws model of the Task 1.1.1 - Phase 10 shake table test structure is shown in Fig. 2-9 and the assigned properties for each shear wall spring element are given in Table 2-3. Using this model the predicted fundamental frequency of the structure is 6.95 Hz. At the start of Phase 10 testing the experimentally measured fundamental frequency was 6.49 Hz. The SAWS model over predicts the experimental result by 7%. Potential sources for this difference are the inherent simplicity of the SAWS model and the estimation of the initial stiffness of each shear wall element.

Figures 2-15 through 2-18 present the SAWS model predictions of the relative displacement and absolute acceleration time-histories along the centerline of the test structure at roof level for Phase 10 – level 4R and 5 testing along with the corresponding experimental results. These relative displacement and absolute acceleration measurements correspond to sensors C16.
and D16, as shown in Fig. 2-6. As with Phase 9, the assigned equivalent viscous damping of 1% of critical in the first and second modes of vibration was used for the analysis of Phase 10 levels 4R and 5.

With respect to maximum response values, over level 4R, the maximum relative displacement and absolute acceleration at sensor location C16 and D16 estimated by the SAWS model under and over predict the experimental values by 10.2% and 8.2%, respectively. Over level 5, the maximum relative displacement and absolute acceleration at sensor location C16 and D16 estimated by the SAWS model over predict the experimental values by 6.4% and 0%, respectively. Table 2-5 provides a complete listing of the maximum response values predicted by the SAWS model and obtained experimentally for all of the sensor locations shown in Fig. 2-6.

All of the dynamic time history results for Phase 10 were produced by the SAWS program using a time integration step of 0.001 sec. In order to properly capture the evolution of damage to the structure over the Phase 10 testing regime the SAWS program was run using the train of shake table acceleration input records from level 4 to 5. It was noted from the experimental results that very little damage occurred to the Phase 10 test structure below level 4. The numerical analysis was performed with a specified tolerance of 5% on the error in the energy balance applied to the equations of motion. For Phase 10, the computational time required to perform a given level of analysis using the SAWS program on a modern desktop computer was less than a minute.
Figure 2-15: Phase 10 – Level 4R Relative Displacement Time Histories at Sensor C16:
   a) SAWS Prediction
   b) Experimental Result
Figure 2-16: Phase 10 – Level 4R Absolute Acceleration Time Histories at Sensor D16:
   a) SAWS Prediction
   b) Experimental Result
Figure 2-17: Phase 10 – Level 5 Relative Displacement Time Histories at Sensor D16:

a) SAWS Prediction
b) Experimental Result
A Computer Program for Seismic Analysis of Woodframe Structures

Figure 2-18: Phase 10 – Level 5 Absolute Acceleration Time Histories at Sensor D16:
   a) SAWS Prediction
   b) Experimental Result
Table 2-5: Hysteretic Parameters for the Shear Wall Spring Elements in Task 1.1.1 - Phase 10 Test Structure.

<table>
<thead>
<tr>
<th>Sensor Locations</th>
<th>Level 4R – Maximum Relative Displacement (mm)</th>
<th>Level 4R – Maximum Absolute Acceleration (g)</th>
<th>Level 5 – Maximum Relative Displacement (mm)</th>
<th>Level 5 – Maximum Absolute Acceleration (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C14</td>
<td>C16</td>
<td>C18</td>
<td>D14</td>
</tr>
<tr>
<td>SAWS Model</td>
<td>13.0</td>
<td>14.1</td>
<td>15.3</td>
<td>1.20</td>
</tr>
<tr>
<td>Test Result</td>
<td>14.5</td>
<td>15.7</td>
<td>16.0</td>
<td>1.00</td>
</tr>
<tr>
<td>% Error</td>
<td>-10.3</td>
<td>-10.2</td>
<td>-4.4</td>
<td>20.0</td>
</tr>
</tbody>
</table>

As noted previously, a static pushover test was not performed on the test structure. However, from the seismic test data from Phase 10, a capacity spectrum was generated as shown in Fig. 2-19. Also shown in Fig. 2-19 are the SAWS model results from three static pushover analyses, corresponding to lateral load distributions that are uniform, triangular and modal adaptive. These three results are in very close agreement with each other and predict a maximum
base shear for the Phase 10 test structure of approximately 130 kN. As noted previously, it is to be expected that the static pushover predictions will be less than the maximum base shear estimated by a capacity spectrum. Also, for comparative purposes, design estimates of maximum base shear are also shown in Fig. 2-19.

Figure 2-19: Phase 10 Experimental Capacity Spectrum and SAWS Pushover Predictions.
2.5. CONCLUDING REMARKS

The main objective of this research project is the formulation and development of a simple numerical model to predict the dynamic characteristics, quasi-static pushover and seismic response of woodframe buildings. The formulation of this numerical model has been presented in detail in Part 1 of this report. This numerical model has been incorporated into the computer program SAWS: Seismic Analysis of Woodframe Structures. The implementation and validation of the SAWS program has been the main focus of this part of the report. In consideration of these matters, the CUREE-Caltech Woodframe Project Task 1.1.1 – Phase 9 and 10 shake table test structures have been modeled with the SAWS program and numerical predictions of the dynamic characteristics, pushover capacities and seismic responses of these structures have been made and compared against the experimental results.

For the Phase 9 bare woodframe structure, it was shown that the CASHEW (Cyclic Analysis of SHEar Walls) program can be used directly to calibrate the shear wall spring elements in the SAWS model. In this case the only experimental data that is required to characterize the response of the whole building structure are the hysteretic properties of the sheathing-to-framing connectors used in the shear walls. For the Phase 10 test structure, which includes both exterior (stucco) and interior (gypsum wallboard) wall finishes, the strength and stiffness representation of these elements within the SAWS model were obtained through a calibration procedure using available experimental data from full-scale wall tests. These two examples illustrate the flexibility and ease in implementing the SAWS program to model woodframe structures with or without wall finishes.
The predictive capabilities of the SAWS program were demonstrated using the Phase 9 and 10 test structures. It was shown that the numerical predictions on the dynamic characteristics, pushover capacities and seismic responses of these structures were generally in good agreement with the experimentally obtained results. For example, the maximum relative displacement predictions were, on average, within 10% of the experimentally measured responses. The maximum absolute acceleration predictions were typically not as good as the relative displacement predictions.

Limitations in the model were also noted. For example, the assumed rigid behavior of the diaphragms in the SAWS model was a contributing factor to the difference between the numerical predictions and the test results for the Phase 9 structure. This influence was less pronounced for the Phase 10 structure. On a positive note, this modeling assumption allows the response of each floor in a building structure to be completely characterized by only three-degrees-of-freedom. This, in turn, permits extremely fast computational turn-around time by the SAWS program. This requirement and ease of implementation were key objectives in the development of the SAWS program. It is believe that the SAWS program has met these objectives of the research and that this simple numerical model can be a useful computational tool for practicing engineers and researchers to analyze the dynamic characteristics, quasi-static pushover and seismic response of woodframe buildings.
2-6. REFERENCES FOR PART 2


PART 3

SAWS PROGRAM USER MANUAL
3-1. INTRODUCTION

The SAWS computer program numerically evaluates the dynamic characteristics, quasi-static pushover and seismic response of woodframe buildings. The theory underlying the development of this program has been presented in Part 1 of this report. The SAWS program has three analysis options:

1. Linear time-history dynamic analysis under bi-directional seismic input.
2. Nonlinear time-history dynamic analysis under bi-directional seismic input.
3. Quasi-static unidirectional pushover analysis.

Part 3 of this report outlines the specifications of the SAWS computer program followed by detailed instructions for creating an input data file to run SAWS. Also included is a sample data file.

3-2. SAWS PROGRAM SPECIFICATIONS

The SAWS computer program has been written in FORTRAN 77 and compiled to run on a microcomputer under the Microsoft Windows® operating system. Before initiating the program, the user first creates a text file containing the input data following the instructions given in Section 3-2.4 of this report. Execution of the SAWS program can be initiated in a number of different ways:

- It can be done within an MS-DOS window by typing SAWS on the command line (this assumes one has assigned the appropriate PATH to the SAWS.EXE file);
- Alternatively one can simply double-click on the SAWS.EXE file using Windows Explorer;
Or one can associate an icon with the SAWS.EXE file and place it on the
Windows Desktop for easy access.

Once SAWS has been executed the user is prompted for the location and name of the data file. The data file name and associate path must meet the operating system requirements. In particular, the user-specified data file name \textit{filename.dat} must have at most 12 alphanumeric characters and must include the .\textit{dat} extension. The combination of the path and data file name cannot exceed 60 characters in total and must not contain any blank spaces. With the name of the data file entered SAWS performs the analysis according to the instructions specified in the data file. Upon completion of the analysis SAWS writes to disk the following output files in the same directory location as the data file:

- \textit{filename.out} Echoes the data input and provides a summary of the key results obtained from the analysis.

- \textit{filename.eqr} Lists the scaled earthquake record used in the analysis. Data output is in two columns: elapsed time is given in column 1 and the corresponding scaled ground acceleration is in column 2 for unidirectional input and columns 2 and 3 for bi-directional input. This file is only generated for analysis options 1 and 2.

- \textit{filename.dis} Records the relative displacement time history of each wall element in the building model. Data output is in multiple columns: elapsed time in the analysis is listed in column 1 and the corresponding relative displacements of the wall elements are recorded in the subsequent columns. Wall elements parallel to the x-axis are listed first followed by those parallel to the y-axis. This file is only generated for analysis options 1 and 2.

- \textit{Filename.acc} Records the absolute acceleration time history of each wall element in the building model. Data output is in multiple columns: elapsed time in the analysis is listed in column 1 and the corresponding absolute accelerations of the wall elements are recorded in the subsequent columns. Wall elements parallel to the x-axis are listed first followed by those parallel to the y-axis. This file is only generated for analysis options 1 and 2.

- \textit{Filename.eng} Records the energy time history for the building model. Data output is in multiple columns: the elapsed time is recorded in column 1 with the
corresponding kinetic, viscous damping, hysteretic and input energies in the subsequent columns. The final column records the error in the energy balance calculation.

**Filename.hys**  
Records the force-deformation response time history of each wall element in the building model. Data output is in multiple pairs of columns. Each pair of columns records the displacement and force response of each wall element. Wall elements parallel to the x-axis are listed first followed by those parallel to the y-axis.

**Filename.poa**  
Records the pushover response of the building model. Data output is in two columns: the top-of-roof displacement is listed in column 1 and the corresponding base shear is recorded in column 2. This file is only generated for analysis option 3.

As compiled, the SAWS computer program has the following input data limitations on the size of problem that can be analyzed:

- Maximum number of rigid diaphragms (MFLR) = 5
- Maximum number of coordinates to define a diaphragm shape (MFC) = 50
- Maximum number of discrete masses in the building model (MDMAS) = 200
- Maximum number of shear wall spring elements in the building model (MWAL) = 200
- Maximum number of time steps in an analysis (MDAT) = 500000

These size limitations can be relaxed by changing a number of the PARAMETER statements in the source code.

**3-3. SAWS DATA FILE – GENERAL INPUT PROCEDURES**

A data file, defining the problem to be analyzed, must be created in advance of running the SAWS program. The user will be prompted by SAWS for the name of this data file. Specific instructions for the creation of this data file are given in the subsequent section. General conventions applying to data input are first discussed.
### 3-3.1. Data Format

All input data required by the SAWS program is read under free-format control. The field definition or delimiter between data entries is one or more blank spaces or a comma. If an isolated exclamation mark is included at the end of a data line all information following the exclamation mark is ignored. Using the exclamation mark allows the user to include comments in the data file.

In the detailed instructions for data input which are given in the next section, the required contents of each input line are contained within a box, followed by a description of the data as illustrated below:

<table>
<thead>
<tr>
<th>IDATA</th>
<th>RDATA</th>
<th>CDATA</th>
</tr>
</thead>
</table>

**IDATA**  
IDATA is the variable name. The variable is of integer type. Variables of this type have names beginning with the letter I-N. Data input in this case is simply an integer value (*i.e.* 126).

**RDATA**  
RDATA is the variable name. The variable is of real type. Variables of this type have names beginning with the letter A-H or O-Z. Data input in this case can be given in either fixed format (*i.e.* 273.34) or exponential format (*i.e.* 2.7334E+02).

**CDATA**  
CDATA is a character string. Data input in this case can consist of alphanumeric characters. Data input of this type is limited to 72 characters in total.
3-3.2. Consistent Units

In creating a data file for subsequent analysis by the SAWS program any consistent set of units can be used. Examples are kilo-Newton (kN) and millimeters for metric units and kips (k) and inches (in.) for US customary units.

3-3.3. Numbering of Components

Data associated with multiple shear wall spring elements and diaphragms must be entered sequentially, starting with component 1.

3-3.4. Overview of the Data Input for SAWS

An index to the detailed description of data input required by the SAWS program to analyze a particular building configuration is presented below:

<table>
<thead>
<tr>
<th>Section</th>
<th>Description of Data Input</th>
</tr>
</thead>
<tbody>
<tr>
<td>3-4.1.</td>
<td>Title for the Analysis.</td>
</tr>
<tr>
<td>3-4.2.</td>
<td>Analysis Control Parameter.</td>
</tr>
<tr>
<td>3-4.3.</td>
<td>Building Configuration.</td>
</tr>
<tr>
<td>3-4.4.</td>
<td>Diaphragm Properties.</td>
</tr>
<tr>
<td>3-4.5.</td>
<td>Discrete Masses.</td>
</tr>
<tr>
<td>3-4.7.</td>
<td>Specification of Walls Aligned Parallel to the Global Y-Axis.</td>
</tr>
<tr>
<td>3-4.8.</td>
<td>Viscous Damping Parameters.</td>
</tr>
<tr>
<td>3-4.9.</td>
<td>Time Integration and Input Ground Acceleration Parameters.</td>
</tr>
</tbody>
</table>
3-4. SAWS DATA FILE - INSTRUCTIONS

3-4.1. Title for the Analysis

Description of the building being analyzed (up to 72 characters in length)

3-4.2. Analysis Control Parameter

<table>
<thead>
<tr>
<th>IANALY</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Data is checked and echoed to file <em>filename.out</em>, after which program execution is terminated.</td>
</tr>
<tr>
<td>1</td>
<td>A linear dynamic time history analysis is performed on the building model.</td>
</tr>
<tr>
<td>2</td>
<td>A nonlinear dynamic time history analysis is performed on the building model.</td>
</tr>
<tr>
<td>3</td>
<td>A quasi-static pushover analysis is performed on the building model.</td>
</tr>
</tbody>
</table>

Note: For all of the analysis options the dynamic characteristics of the structure are determined.

3-4.3. Building Configuration

<table>
<thead>
<tr>
<th>NFLOOR</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Number of rigid diaphragms (floors) in the building model. (NFLOOR ≤ 5).</td>
</tr>
</tbody>
</table>

Note: The foundation is not included in the count of rigid diaphragms.
3-4.4. Diaphragm Properties

The two input steps in Section 3-4.4, as a block, must be repeated NFLOOR times.

One input line is required for each floor diaphragm.

<table>
<thead>
<tr>
<th>IFLR</th>
<th>NFC</th>
<th>FMASS</th>
<th>FELV</th>
</tr>
</thead>
<tbody>
<tr>
<td>IFLR</td>
<td>NFC</td>
<td>FMASS</td>
<td>FELV</td>
</tr>
</tbody>
</table>

  IFLR     Floor diaphragm number.
  NFC      Number of coordinates to define the diaphragm shape.
  FMASS    Mass of floor diaphragm. A uniform distribution of mass over the diaphragm area is assumed.
  FELV     Elevation of diaphragm measured with respect to the foundation as the zero datum.

One input line is required for each defining coordinate of the floor diaphragm. Input continues in turn for each coordinate up to NFC.

<table>
<thead>
<tr>
<th>XFC</th>
<th>YFC</th>
</tr>
</thead>
<tbody>
<tr>
<td>XFC</td>
<td>YFC</td>
</tr>
</tbody>
</table>

  XFC      X-coordinate of the floor diaphragm.
  YFC      Y-coordinate of the floor diaphragm.
Figure 3-1: Layout of a Typical Diaphragm.

Notes:

1. With respect to global x- and y-axes, the entire building model must lie in the first quadrant.
2. The diaphragm can be any polygonal shape, with the restriction that each side must run parallel to either the x-axis or y-axis.
3. The defining coordinates of each diaphragm must be entered in such a manner as to draw the diaphragm in a counterclockwise direction. For the diaphragm shown above this can be accomplished by sequentially entering the coordinates \((x_1,y_1)\) through to \((x_6,y_6)\).
3-4.5. Discrete Masses

NDMASS

NDMASS  Number of discrete masses located throughout the structure (NDMASS ≤ 200).

One input line is required for each discrete mass.

IFDM  DMX  DMY  DMASS  DMROT

IFDM    Floor level of the discrete mass.
DMX    Global x-coordinate of the discrete mass on the given floor.
DMY    Global y-coordinate of the discrete mass on the given floor.
DMASS    Translational mass associated with discrete mass element.
DMROT    Rotational mass associated with discrete mass element.

Note: Specification of discrete masses can be used to account for a non-uniform distribution of mass over a diaphragm and/or to represent the mass of wall elements in the building model.

3-4.6. Specification of Walls Aligned Parallel to the Global X-Axis

NXWALL

NXWALL  Number of shear spring elements used to model walls in the building that are aligned parallel to the global x-axis (NXWALL ≤ 100).

The next three input steps in Section 3-4.6, as a block, must be repeated NXWALL times.

One input line is required for each shear wall spring element.

YXWALL  ICONXW(1)  ICONXW(2)

YXWALL    Y-coordinate of the shear wall spring element.
ICONXW(1)    Floor level of lower node of the shear wall spring element.
ICONXW(2)    Floor level of upper node of the shear wall spring element.
Figure 3-2: Building Layout, Showing Placement and Connectivity of Shear Wall Spring Elements. This Building Model Represents a Two-Story Structure.
One input line is required for each shear wall spring element.

<table>
<thead>
<tr>
<th>F0</th>
<th>FI</th>
<th>DU</th>
</tr>
</thead>
<tbody>
<tr>
<td>F0</td>
<td>Intercept strength of the shear wall spring element for the asymptotic line to the envelope curve. ((F0 &gt; FI &gt; 0)).</td>
<td></td>
</tr>
<tr>
<td>FI</td>
<td>Intercept strength of the spring element for the pinching branch of the hysteretic curve. ((FI &gt; 0)).</td>
<td></td>
</tr>
<tr>
<td>DU</td>
<td>Spring element displacement at ultimate load. ((DU &gt; 0)).</td>
<td></td>
</tr>
</tbody>
</table>

One input line is required for each shear wall spring element.

<table>
<thead>
<tr>
<th>S0</th>
<th>R1</th>
<th>R2</th>
<th>R3</th>
<th>R4</th>
</tr>
</thead>
<tbody>
<tr>
<td>S0</td>
<td>Initial stiffness of the shear wall spring element ((S0 &gt; 0)).</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>R1</td>
<td>Stiffness ratio of the asymptotic line to the spring element envelope curve. The slope of this line is (R1\cdot S0). ((0 &lt; R1 &lt; 1.0)).</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>R2</td>
<td>Stiffness ratio of the descending branch of the spring element envelope curve. The slope of this line is (R2\cdot S0). ((R2 &lt; 0)).</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>R3</td>
<td>Stiffness ratio of the unloading branch off the spring element envelope curve. The slope of this line is (R3\cdot S0). ((R3 \geq 1)).</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>R4</td>
<td>Stiffness ratio of the pinching branch for the spring element. The slope of this line is (R4\cdot S0). ((R4 &gt; 0)).</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ALPHA</th>
<th>BETA</th>
</tr>
</thead>
<tbody>
<tr>
<td>ALPHA</td>
<td>Stiffness degradation parameter for the shear wall spring element. ((ALPHA &gt; 0)).</td>
</tr>
<tr>
<td>BETA</td>
<td>Stiffness degradation parameter for the spring element. ((BETA &gt; 0)).</td>
</tr>
</tbody>
</table>
Figure 3-3: Parameters Required to Define the Hysteretic Response of a Shear Wall Spring Element.

3-4.7. Specification of Walls Aligned Parallel to the Global Y-Axis

NYWALL

NYWALL Number of shear spring elements used to model walls in the building that are aligned parallel to the global y-axis (NYWALL ≤ 100).

\[
K_p = S_0 \left[ \frac{(F_0/S_0)/\delta_{\text{max}}}{\delta_{\text{un}}} \right]^\alpha
\]

\[
\delta_{\text{max}} = \beta \cdot \delta_{\text{un}}
\]
The next three input steps in Section 3-4.7, as a block, must be repeated NYWALL times.

One input line is required for each shear wall spring element.

**XYWALL ICONYW(1) ICONYW(2)**

**XYWALL** X-coordinate of the shear wall spring element.
**ICONYW(1)** Floor level of lower node of the shear wall spring element.
**ICONYW(2)** Floor level of upper node of the shear wall spring element.

One input line is required for each shear wall spring element.

**F0 FI DU**

**F0** Intercept strength of the shear wall spring element for the asymptotic line to the envelope curve. \((F0 > FI > 0)\).
**FI** Intercept strength of the spring element for the pinching branch of the hysteretic curve. \((FI > 0)\).
**DU** Spring element displacement at ultimate load. \((DU > 0)\).

One input line is required for each shear wall spring element.

**S0 R1 R2 R3 R4**

**S0** Initial stiffness of the shear wall spring element \((S0 > 0)\).
**R1** Stiffness ratio of the asymptotic line to the spring element envelope curve. The slope of this line is \(R1 \cdot S0\). \((0 < R1 < 1.0)\).
**R2** Stiffness ratio of the descending branch of the spring element envelope curve. The slope of this line is \(R2 \cdot S0\). \((R2 < 0)\).
**R3** Stiffness ratio of the unloading branch off the spring element envelope curve. The slope of this line is \(R3 \cdot S0\). \((R3 \geq 1)\).
**R4** Stiffness ratio of the pinching branch for the spring element. The slope of this line is \(R4 \cdot S0\). \((R4 > 0)\).
One input line is required for each shear wall spring element.

**ALPHA BETA**

ALPHA  Stiffness degradation parameter for the shear wall spring element.  \((\text{ALPHA} > 0)\).
BETA   Stiffness degradation parameter for the spring element.  \((\text{BETA} > 0)\).

### 3-4.8. Viscous Damping Parameters

**MODE1 ZETA1 MODE2 ZETA2**

MODE1  First mode of vibration to assign damping parameter ZETA1.
ZETA1  Fraction of critical viscous damping in MODE1.
MODE2  Second mode of vibration to assign damping parameter ZETA2.
ZETA2  Fraction of critical viscous damping in MODE2.

If \(\text{IANALY} = 3\) then go to Section 3-4.1 1.

### 3-4.9. Time Integration and Input Ground Acceleration Parameters

**DELTA INTER NSTEP TOLER NPDATA ISCALE ACCMAX IEQXY**

DELTA  Time step increment used in the dynamic analysis.
INTER  Interval for print out of dynamic analysis output, expressed as a multiple of the time step DELTA.
NSTEP  Number of integration time steps to be performed.
TOLER  Tolerance on energy balance calculation in % (typically 5%).
NPDATA Number of time-acceleration pairs defining the input ground motion.
ISCALE  = 0  Input acceleration record is not scaled.
         = 1  Input acceleration record is to be scaled to produce a peak ground acceleration of ACCMAX in absolute value.
ACCMAX Peak ground acceleration.  If \(\text{ISCALE} = 0\) then ACCMAX is set equal to 1.
IEQXY  = 1  Input ground acceleration record is applied parallel to the global x-axis of the building model.
        = 2  Input ground acceleration record is applied parallel to the global y-
axis of the building model.

Input ground acceleration record is applied parallel to both the global x-axis and y-axis of the building model. In this case ISCALE is to be set to 0.

3-4.10. Input Ground Acceleration Record

If IEQXY = 1 or 2 then enter data as follows. One input line is required for each time-acceleration pair in the input ground acceleration record. Input continues up to NPDATA times to enter the acceleration record.

<table>
<thead>
<tr>
<th>TDAT</th>
<th>ADAT</th>
</tr>
</thead>
</table>

TDAT  Time associated with input ground acceleration value ADAT.
ADAT  Input ground acceleration value at time TDAT. If ISCALE = 1, ADAT is not unit dependent since the input ground acceleration record is scaled by the factor ACCMAX. If ISCALE = 0, ADAT must have consistent units.

If IEQXY = 3 then enter data as follows. One input line is required for each time-acceleration trio in the input ground acceleration record. Input continues up to NPDATA times to enter the acceleration record.

<table>
<thead>
<tr>
<th>TDAT</th>
<th>ADATX</th>
<th>ADATY</th>
</tr>
</thead>
</table>

TDAT  Time associated with input ground acceleration values ADATX and ADATY.
ADATX Input ground acceleration value parallel to the x-axis at time TDAT. Note in this case ISCALE is set to 0. Therefore, ADATX is unscaled and must have consistent units.
ADATY Input ground acceleration value parallel to the y-axis at time TDAT. Note in this case ISCALE is set to 0. Therefore, ADATY is unscaled and must have consistent units.

If IANALY = 1 or 2 then data input is complete.
3-4.11. Pushover Analysis Parameters

<table>
<thead>
<tr>
<th>IPOTYP</th>
<th>IPOXY</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>= 1</td>
<td></td>
<td>A pushover analysis is performed utilizing a uniform lateral load distribution over the building height.</td>
</tr>
<tr>
<td>= 2</td>
<td></td>
<td>A pushover analysis is performed utilizing an inverted triangular lateral load distribution over the building height.</td>
</tr>
<tr>
<td>= 3</td>
<td></td>
<td>A pushover analysis is performed utilizing a modal adaptive lateral load distribution over the building height.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>IPOXY</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>= 1</td>
<td>The lateral loads are applied parallel to the global x-axis of the building model.</td>
</tr>
<tr>
<td>= 2</td>
<td>The lateral loads are applied parallel to the global y-axis of the building model.</td>
</tr>
<tr>
<td>= 3</td>
<td>The lateral loads are applied parallel to both the global x-axis and y-axis of the building model.</td>
</tr>
</tbody>
</table>

If IANALY = 3 then data input is complete.
3-5. **SAWS EXECUTION ERRORS AND PROGRAM TERMINATION**

The SAWS program will capture various data input errors at run-time avoiding an execution error within the program. For example, the SAWS program will check the building configuration against the size limitations noted in Section 3-2 and will alert the user of the problem before an execution error occurs. In addition, data checks are performed on shear wall spring element properties. Any data errors that are identified result in descriptive error messages being written to the output file `filename.out`. The error checking process in the SAWS program is not exhaustive and execution errors may occur.

The SAWS program may also prematurely terminate an analysis for one of the following reasons:

1. Exceeding the energy balance tolerance;
2. Having the load-carrying capacity of the building exhausted by the applied loading;
3. Having the global stiffness matrix become singular;
4. Encountering too large of a displacement step (in a given time-step) for a shear spring element to identify the correct load-deformation path to follow.

If any of these run-time errors occur an appropriate error message will be printed to the screen and the SAWS program will terminate the analysis. Possible reasons for premature program termination may be linked to incorrect data input or to not specifying a sufficiently small time-step (particularly under analysis option `IANALY = 2`).
3-6. EXAMPLE SAWS DATA FILE AND SUMMARY OUTPUT FILE

The following data file was created for the pushover analysis of the Task 1.1.1 – Phase 9 test structure described in Part 2 - Section 3 and 4 of this report. With this data file the test structure is subjected to a pushover analysis in the x-direction under a uniform lateral load distribution (IANALY = 3; IPOXY = 1; IPOTYP = 1). Note that for this data file the units are kips and inches.

EXAMPLE.DAT

CUREE-CalTech Woodframe House Phase 9 (Units: kips & in)
3,  ! Analysis option
2,  ! Number of floors
1.4,0.03580,108.0,  ! Floor # 1 geometry, mass & elevation
0.0,0.0,  ! Floor # 1 coordinates
192.0,0.0,  ! Floor # 1 coordinates
192.0,240.0,  ! Floor # 1 coordinates
0.0,240.0,  ! Floor # 1 coordinates
0,  ! Number of discrete masses
4,  ! Number of walls parallel to x-axis
8.224,1.880,3.437,  ! Hysteretic parameters for wall # 1
16.75,0.08254,-0.08805,1.000,0.03302,  ! Hysteretic parameters for wall # 1
0.7856,1.073,  ! Hysteretic parameters for wall # 1
240.0,0,1,  ! Location and connectivity of wall #2
8.258,1.839,2.275,  ! Hysteretic parameters for wall # 2
22.21,0.06358,-0.05599,1.161,0.02259,  ! Hysteretic parameters for wall # 2
0.8731,1.112,  ! Hysteretic parameters for wall # 2
0.0,1,2,  ! Location and connectivity of wall #3
4.409,1.070,3.024,  ! Hysteretic parameters for wall # 3
11.99,0.06868,-0.03793,1.161,0.02259,  ! Hysteretic parameters for wall # 3
0.7680,1.101,  ! Hysteretic parameters for wall # 3
240.0,1.2,  ! Location and connectivity of wall #4
4.409,1.070,3.024,  ! Hysteretic parameters for wall # 4
11.99,0.06868,-0.03793,1.161,0.02259,  ! Hysteretic parameters for wall # 4
0.7680,1.101,  ! Hysteretic parameters for wall # 4
4,  ! Number of walls parallel to y-axis
0.0,0.1,  ! Location and connectivity of wall #1
The associated summary output file produced by SAWS is given below:

EXAMPLE.OUT

------------------------------------------------------------------------
Program SAWS: Seismic Analysis of Woodframe Structures
   (Version 1.0)

Written by: Bryan Folz, Ph.D.
Structural Engineering Department
University of California, San Diego
       June 2001
------------------------------------------------------------------------

Problem: CUREE-CalTech Woodframe House Phase 9 (Units: kips & in)

Solution option: Static push-over analysis.

Number of floors (rigid diaphragms) = 2
Number of degrees-of-freedom = 6
Total number of shear walls elements = 8
   - Number parallel to global x-axis = 4
   - Number parallel to global y-axis = 4

Diaphragm properties:
   - Floor number = 1
     - Floor mass = 0.35800E-01
     - Floor elevation = 0.10800E+03
     - Floor coordinates:
       ( 0.00000E+00,  0.00000E+00)    ( 0.19200E+03,  0.00000E+00)
       ( 0.19200E+03,  0.24000E+03)    ( 0.00000E+00,  0.00000E+00)
   - Floor number = 2
     - Floor mass = 0.27800E-01
     - Floor elevation = 0.21600E+03
     - Floor coordinates:
       ( 0.00000E+00,  0.00000E+00)    ( 0.19200E+03,  0.00000E+00)
       ( 0.19200E+03,  0.24000E+03)    ( 0.00000E+00,  0.00000E+00)

Shear wall spring elements aligned parallel to x-axis:
   - X-wall number = 1
     - Y-axis coordinate = 0.00000E+00
     - Floor connection at bottom of wall = 0
     - Floor connection at top of wall = 1
     - Shear wall spring properties:
       Stiffness parameters:
       \[
       S_0 = 0.16750E+02, \quad R_1 = 0.82540E-01, \quad R_2 = -0.88050E-01, \quad R_3 = 0.10000E+01, \quad R_4 = 0.33020E-01
       \]
       Strength, deformation and degradation parameters:
       \[
       F_0 = \text{FO}, \quad F_I = \text{FI}, \quad D_U = \text{DU}, \quad A_LPHA = \text{ALPHA}, \quad B_ETA = \text{BETA}
       \]
| 0.82240E+01 | 0.18800E+01 | 0.34370E+01 | 0.78560E+00 | 0.10730E+01 |
| X-wall number = 2 |
| Y-axis coordinate = 0.24000E+03 |
| Floor connection at bottom of wall = 0 |
| Floor connection at top of wall = 1 |
| Shear wall spring properties: |
| Stiffness parameters: |
| S0 | R1 | R2 | R3 | R4 |
| 0.22210E+02 | 0.63580E-01 | -0.55990E-01 | 0.10720E+01 | 0.30060E-01 |
| Strength, deformation and degradation parameters: |
| F0 | FI | DU | ALPHA | BETA |
| 0.82580E+01 | 0.18390E+01 | 0.22750E+01 | 0.87310E+00 | 0.11120E+01 |

| - X-wall number = 3 |
| - Y-axis coordinate = 0.00000E+00 |
| - Floor connection at bottom of wall = 1 |
| - Floor connection at top of wall = 2 |
| - Shear wall spring properties: |
| Stiffness parameters: |
| S0 | R1 | R2 | R3 | R4 |
| 0.11990E+02 | 0.68680E-01 | -0.37930E-01 | 0.11610E+01 | 0.22590E-01 |
| Strength, deformation and degradation parameters: |
| F0 | FI | DU | ALPHA | BETA |
| 0.44090E+01 | 0.10700E+01 | 0.30240E+01 | 0.76800E+00 | 0.11010E+01 |

| - X-wall number = 4 |
| - Y-axis coordinate = 0.24000E+03 |
| - Floor connection at bottom of wall = 1 |
| - Floor connection at top of wall = 2 |
| - Shear wall spring properties: |
| Stiffness parameters: |
| S0 | R1 | R2 | R3 | R4 |
| 0.11990E+02 | 0.68680E-01 | -0.37930E-01 | 0.11610E+01 | 0.22590E-01 |
| Strength, deformation and degradation parameters: |
| F0 | FI | DU | ALPHA | BETA |
| 0.44090E+01 | 0.10700E+01 | 0.30240E+01 | 0.76800E+00 | 0.11010E+01 |

Shear wall spring elements aligned parallel to y-axis:

| - Y-wall number = 1 |
| - X-axis coordinate = 0.00000E+00 |
| - Floor connection at bottom of wall = 0 |
| - Floor connection at top of wall = 1 |
| - Shear wall spring properties: |
| Stiffness parameters: |
| S0 | R1 | R2 | R3 | R4 |
| 0.32480E+02 | 0.64940E-01 | -0.73740E-01 | 0.10980E+01 | 0.28730E-01 |
| Strength, deformation and degradation parameters: |
| F0 | FI | DU | ALPHA | BETA |
| 0.10870E+02 | 0.24370E+01 | 0.23810E+01 | 0.80740E+00 | 0.10873E+01 |

| - Y-wall number = 2 |
| - X-axis coordinate = 0.19200E+03 |
| - Floor connection at bottom of wall = 0 |
| - Floor connection at top of wall = 1 |
| - Shear wall spring properties: |
| Stiffness parameters: |
| S0 | R1 | R2 | R3 | R4 |
| 0.32480E+02 | 0.64940E-01 | -0.73740E-01 | 0.10980E+01 | 0.28730E-01 |
Strength, deformation and degradation parameters:

<table>
<thead>
<tr>
<th>F0</th>
<th>FI</th>
<th>DU</th>
<th>ALPHA</th>
<th>BETA</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.10870E+02</td>
<td>0.24370E+01</td>
<td>0.23810E+01</td>
<td>0.80740E+00</td>
<td>0.10873E+01</td>
</tr>
</tbody>
</table>

- Y-wall number = 3
- X-axis coordinate = 0.00000E+00
- Floor connection at bottom of wall = 1
- Floor connection at top of wall = 2

Shear wall spring properties:
Stiffness parameters:

<table>
<thead>
<tr>
<th>S0</th>
<th>R1</th>
<th>R2</th>
<th>R3</th>
<th>R4</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.19130E+02</td>
<td>0.54000E-01</td>
<td>-0.60000E-01</td>
<td>0.11000E+01</td>
<td>0.30000E-01</td>
</tr>
</tbody>
</table>

Strength, deformation and degradation parameters:

<table>
<thead>
<tr>
<th>F0</th>
<th>FI</th>
<th>DU</th>
<th>ALPHA</th>
<th>BETA</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.79360E+01</td>
<td>0.29000E+01</td>
<td>0.29090E+01</td>
<td>0.84000E+00</td>
<td>0.10900E+01</td>
</tr>
</tbody>
</table>

- Y-wall number = 4
- X-axis coordinate = 0.19200E+03
- Floor connection at bottom of wall = 1
- Floor connection at top of wall = 2

Shear wall spring properties:
Stiffness parameters:

<table>
<thead>
<tr>
<th>S0</th>
<th>R1</th>
<th>R2</th>
<th>R3</th>
<th>R4</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.19130E+02</td>
<td>0.54000E-01</td>
<td>-0.60000E-01</td>
<td>0.11000E+01</td>
<td>0.30000E-01</td>
</tr>
</tbody>
</table>

Strength, deformation and degradation parameters:

<table>
<thead>
<tr>
<th>F0</th>
<th>FI</th>
<th>DU</th>
<th>ALPHA</th>
<th>BETA</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.79360E+01</td>
<td>0.29000E+01</td>
<td>0.29090E+01</td>
<td>0.84000E+00</td>
<td>0.10900E+01</td>
</tr>
</tbody>
</table>

Viscous damping parameters:

- Mode to assign 1st damping parameter = 1
- Fraction of critical damping = 0.10000E-01
- Mode to assign 2nd damping parameter = 2
- Fraction of critical damping = 0.10000E-01

Energy balance is recorded in the file: push9.ENG

Hysteretic response of each wall is recorded in the file: push9.HYS

Push-over analysis is recorded in the file: push9.POA

Frequency Analysis:

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency</th>
<th>Period</th>
<th>% Damping</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.3281E+01</td>
<td>0.3048E+00</td>
<td>0.1000E+01</td>
</tr>
<tr>
<td>2</td>
<td>0.4217E+01</td>
<td>0.2371E+00</td>
<td>0.1000E+01</td>
</tr>
<tr>
<td>3</td>
<td>0.6355E+01</td>
<td>0.1574E+00</td>
<td>0.1138E+01</td>
</tr>
<tr>
<td>4</td>
<td>0.7467E+01</td>
<td>0.1339E+00</td>
<td>0.1243E+01</td>
</tr>
<tr>
<td>5</td>
<td>0.9493E+01</td>
<td>0.1053E+00</td>
<td>0.1460E+01</td>
</tr>
<tr>
<td>6</td>
<td>0.1439E+02</td>
<td>0.6947E-01</td>
<td>0.2048E+01</td>
</tr>
</tbody>
</table>
Modes Shapes:

<table>
<thead>
<tr>
<th>DOF</th>
<th>Modes</th>
<th>DOF</th>
<th>Modes</th>
<th>DOF</th>
<th>Modes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.524E+00  -0.108E-13</td>
<td>2</td>
<td>-0.326E-01</td>
<td>0.490E+00</td>
<td>-0.442E+00</td>
</tr>
<tr>
<td>3</td>
<td>0.340E-03</td>
<td>-0.626E-17</td>
<td>4</td>
<td>0.100E+01</td>
<td>-0.206E-13</td>
</tr>
<tr>
<td>5</td>
<td>0.392E-03</td>
<td>-0.629E-17</td>
<td>6</td>
<td>0.100E+01</td>
<td>-0.206E-13</td>
</tr>
</tbody>
</table>

Modal Participation Factors:

<table>
<thead>
<tr>
<th>Mode</th>
<th>MP-Factor</th>
<th>MP-Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X-Shaking</td>
<td>Y-Shaking</td>
</tr>
<tr>
<td>1</td>
<td>0.1300E+01</td>
<td>-0.3452E-13</td>
</tr>
<tr>
<td>2</td>
<td>-0.2503E-13</td>
<td>0.1246E+01</td>
</tr>
<tr>
<td>3</td>
<td>0.1068E-01</td>
<td>-0.1675E-13</td>
</tr>
<tr>
<td>4</td>
<td>0.3410E+00</td>
<td>-0.5445E-13</td>
</tr>
<tr>
<td>5</td>
<td>0.1106E-12</td>
<td>0.3896E+00</td>
</tr>
<tr>
<td>6</td>
<td>-0.2958E-01</td>
<td>0.1382E-15</td>
</tr>
</tbody>
</table>

Summary of Static Push-Over Analysis:

- Lateral loads applied parallel to X-axis.
- Pushover analysis used a uniform lateral load distribution.

Maximum base shear force = 0.2278E+02
Relative roof displacement = 0.4799E+01
Drift at roof level = 0.2222E-01