CUREE Publication No. W-08

A Computer Program for the Cyclic Analysis of Shearwalls in Woodframe Structures

Bryan Folz
André Filiatrault

2002

CUREE

the CUREE-Caltech Woodframe Project

Earthquake Hazard Mitigation of Woodframe Construction

Funded by the Federal Emergency Management Agency through a Hazard Mitigation Grant Program award administered by the California Governor's Office of Emergency Services

Disclaimer

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

the CUREE-Caltech Woodframe Project

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

CUREE Publication No. W-08

A Computer Program for Cyclic Analysis of Shearwall in Woodframe Structures

Bryan Folz
André Filiatrault

Department of Structural Engineering
University of California, San Diego

2002

CUREE
Consortium of Universities for Research in Earthquake Engineering
1301 S. 46th Street
Richmond, CA 94804
tel.: 510-665-3529 fax: 510-665-3622
email: curee@curee.org website: www.curee.org

CUREE

Published by
Consortium of Universities for Research in Earthquake Engineering (CUREE)
1301 S. 46th Street - Richmond, CA 94804-4600
www.curee.org (CUREE Worldwide Website)

DISCLAIMER

ACKNOWLEDGEMENTS

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

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.

The authors also thank Professor Helmut Prion of the University of British Columbia for graciously providing shear wall test data from Jennifer Durham's M.A.Sc. Thesis.

LIST OF SYMBOLS

$\boldsymbol{b}$	Width of sheathing panel
$\boldsymbol{B_s}$	Compatibility vector
$\boldsymbol{D}$	Global displacement vector
$\boldsymbol{F}$	Global force vector
$\boldsymbol{F_I}$	Zero-displacement load intercept of sheathing-framing connector
$\boldsymbol{F_F}$	Lateral force at the top of the wall
$\boldsymbol{F_j}$	Force in sheathing-to-framing connector j
$\boldsymbol{F_0}$	Force intercept of the asymptotic line for the sheathing-to-framing connector
$\boldsymbol{F_u}$	Ultimate load of sheathing-to-framing connector
$\boldsymbol{F_{un}}$	Unloading force in sheathing-to-framing connector in previous cycle
$\boldsymbol{G}$	Shear modulus of sheathing panel
$\boldsymbol{h}$	Height of sheathing panel
$\boldsymbol{H}$	Overall height of wall
$\boldsymbol{k_{ij}^{(s)}}$	Secant stiffness coefficient of the j-th connector in the i-th sheathing panel
$\boldsymbol{k_{ij}^{(T)}}$	Tangent stiffness coefficient of the j-th connector in the i-th sheathing panel
$\boldsymbol{K_0}$	Initial stiffness of sheathing-to-framing connector
$\boldsymbol{K_p}$	Re-loading degrading stiffness of sheathing-to-framing connector
$\boldsymbol{K_S}$	Global secant stiffness matrix
$\boldsymbol{K_T}$	Global tangent stiffness matrix
$\boldsymbol{N_c}$	Number of sheathing-to-framing connectors
$\boldsymbol{N_p}$	Number of sheathing panels

P_u Lateral load carrying capacity of shear wall

Q Generic point on sheathing panel with initial local panel coordinates ***(x,y)***

r_1K_0 Asymptotic stiffness of sheathing-to-framing connector under monotonic load

r_2K_0 Post ultimate strength stiffness of sheathing-to-framing connector under monotonic load

r_3K_0 Unloading stiffness of sheathing-to-framing connector

r_4K_0 Re-loading pinched stiffness of sheathing-to-framing connector

R Global residual force vector

t Load-step

t_p Thickness of sheathing panel

u_p, v_p Linearized deformations under racking of the wall

U_F Lateral displacement at the top of the wall

U_s, In-plane shear deformation of sheathing panel

$\overline{U}$ Horizontal rigid-body translation of sheathing panel

$\overline{V}$ Vertical rigid-body translation of sheathing panel

$\bar{x}, \bar{y}$ Centroidal coordinates of sheathing panel in undeformed configuration

x_{ij}, y_{ij} Coordinates of the j-th connector in the i-th sheathing panel

V_p Volume of sheathing panel

W_C Internal work contribution from all sheathing-to-framing connectors

W_E External work

W_I Total internal work

W_F Internal work contribution from all framing members

W_S Internal work contribution from all sheathing panels

α, β	Hysteretic parameters for stiffness degradation of sheathing-to-framing connector
δ	Deformation of sheathing-to-framing connector
δ_F	Deformation of sheathing-to-framing connector at failure
δ_j	Deformation of sheathing-to-framing connector j
δ_{max}	Maximum deformation of sheathing-to-framing connector at a given cycle
δ_u	Deformation of sheathing-to-framing connector at ultimate load
δ_{un}	Unloading deformation of sheathing-to-framing connector in previous cycle
δ_u	Horizontal component of deformation in sheathing-to-framing connector
δ_v	Vertical component of deformation in sheathing-to-framing connector
Δ	Reference deformation of CUREe-Caltech loading protocol
Δ_m	Deformation of a shear wall at which the applied load drops, for the first time, below 80% of the maximum load that was applied to the wall
Δt	Load-step increment
Δ_u	Shear wall displacement at ultimate load
γ	Shear strain field in sheathing panel
λ	Load factor applied to reference global load vector
Θ	Rigid-body rotation of sheathing panel

TABLE OF CONTENTS

TABLE OF CONTENTS

SCOPE OF RESEARCH

The main objective of this research project is to present a formulation for the structural analysis of wood framed shear walls under general cyclic loading. The numerical model, presented herein and integrated in the computer program CASHEW: Cyclic Analysis of wood SHEar Walls, predicts for sheathed shear walls with or without opening the load-displacement response and energy dissipation characteristics under arbitrary quasi-static cyclic loading. In formulating this structural analysis tool a balance has been sought between model complexity and computational overhead. The proposed model is validated against full-scale tests of wood shear walls subjected to monotonic and cyclic loading. It is also shown how this model can be used to calibrate the parameters of an equivalent SDOF hysteretic shear element to predict the global cyclic response of a shear wall.

REPORT LAYOUT

Part 1: CYCLIC ANALYSIS OF SHEAR WALLS – MODEL FORMULATION, VERIFICATION and IMPLEMENTATION

Section 1-1 gives a brief overview of the state of the research in numerically modeling the structural response of wood shear walls. **Section 1-2** describes the underlying theory leading to the numerical formulation of the CASHEW computer program. **Section 1-3** compares the prediction of the CASHEW model against full-scale shear wall monotonic and cyclic tests conducted at the University of British Columbia. **Section 1-4** discusses the loading protocol developed within the CUREe-Caltech Woodframe Project. This loading protocol is included as an automatic loading option in the CASHEW program. **Section 1-5** provides concluding comments on the development of the CASHEW model. Part 1 concludes with a reference section of cited work and an appendix on the evaluation of the global stiffness matrices.

Part 2: CASHEW PROGRAM USER MANUAL

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

PART 1

CYCLIC ANALYSIS OF SHEAR WALLS

MODEL FORMULATION VERIFICATION AND IMPLEMENTATION

1-1. INTRODUCTION

In low-rise wood framed structures subjected to earthquake loading, shear walls are commonly used as the primary component of the lateral load-resisting system. Under such loading, the racking response of a shear wall is generally cyclic in nature. Building codes, however, have typically assigned design shear strength values to wood shear walls based on experimental data obtained from static racking tests. In recent years there has been a proliferation of full-scale testing performed on wood shear walls under cyclic loading. The objective of this increased research activity has been to provide a greater understanding of the response of shear walls under loading conditions that closer resemble the seismic response scenario. Cyclic loading, unlike monotonic loading, is not uniquely defined. Numerous cyclic loading protocols have been proposed (CEN 1995; CoLA/UCI Committee 1999; ISO 1999; Krawinkler et al. 2000), and the performance of wood shear walls against these protocols have been experimentally investigated (He et al. 1998; Rose 1994; Skaggs and Rose 1996). Other experimental studies have focused on the degrading response of shear walls under cyclic loading (Shenton et al. 1998). Still other experimental studies have considered the influence of panel size (Lam et al. 1997), openings (He et al. 1999), fastener type and the contribution from gypsum wall board (Karacabeyli and Ceccotti 1996) and the effect of hold-downs (Commins and Gregg 1994) on the cyclic response of wood shear walls. Even with this extensive effort in experimentally evaluating the cyclic response of wood shear walls there still remains a need for a more complete understanding of these structural elements. It is obvious that this cannot be achieved through testing programs alone; numerical studies should be conducted in parallel to complement this experimental work.

A large number of numerical models, of varying complexity, have been formulated to predict the static racking response of wood shear walls. In the simpler models, the non-linear global wall response is fully attributable to the non-linear load-deformation behavior of the sheathing-to-framing connectors (Tuomi and McCutheon 1978; Gupta and Kuo 1985, 1987, Filiatrault 1990). The sheathing is generally assumed to develop only elastic in-plane shear forces. It was also found that bending of the framing members contributed little to the global wall response (Gupta and Kuo 1985). Consequently, in a number of these studies the framing members are assumed to be rigid. These models generally provide good agreement with the load-displacement response obtained from tests. However, because of their simplicity they are not able to capture the detailed interaction and load sharing between the components of the shear wall under the imposed lateral loading.

More sophisticated finite element models have also been proposed (Itani and Cheung 1984; Gutkowski and Castillo 1988; Dolan and Foschi 1991; White and Dolan 1995). In these models the framing members are comprised of beam elements and the sheathing is represented by plane stress elements or plate bending elements. The sheathing-to-framing connectors are modeled using springs with non-linear load-deformation characteristics. Also, gap and bearing elements have been included along the interface between the sheathing panels. Obviously, these models are able to capture more fully the inter-component response within the wall. However, with this increased model complexity a greater computational effort must be expended. Interestingly, the overall global load-displacement predictions of these models produce essentially the same level of correlation with experimental data as the simpler models discussed previously.

Several non-linear dynamic analysis models have also been developed for predicting the seismic response of wood shear walls. The simplest of these are single degree-of-freedom (SDOF) lumped parameter models (*e.g*. Stewart 1987; Foliente 1995). These, however, have limited application because the model must be calibrated, in each case, to full-scale test data. Others have extended existing static shear wall models to perform non-linear dynamic time history analysis under seismic input (Dolan 1989; Filiatrault 1990).

Between the bookends of static ultimate load analysis and the full non-linear dynamic analysis lies the cyclic analysis of shear walls. As mentioned previously, it is this loading regime that has received the greatest experimental research attention of late. Surprisingly, however, very little research has been applied to developing structural analysis models that can complement this experimental effort.

1-2. NUMERICAL MODEL FORMULATION

1-2.1. Structural Configuration

Typical wood shear wall assemblies are comprised of 4 basic structural components, as shown in Fig. 1: framing members, sheathing panels, sheathing-to-framing connectors and hold-down anchorage devices. The framing members are generally sawn lumber pieces oriented horizontally (plates and sills) and vertically (studs) with only nominal nailing to hold the framework together. Sheathing panels are usually made of plywood, oriented strand board or other structural panel products. These panels may be applied to one or both sides of the wall. Openings in the paneling may occur to facilitate placement of windows or doors. In such cases, additional framing members are required to reinforce these openings. Sheathing-to-framing connectors are most commonly dowel type fasteners such as nails. These fasteners are typically spaced at regular intervals with fastener lines around the perimeter of the sheathing panels more densely spaced than throughout the panel interior. Hold-down anchorage devices, in the form of sill nails, simple anchor bolts and proprietary hardware, may be included in the wall assembly. The sill nails and/or anchor bolts principally function to transfer shear from the wall to the supporting floor or foundation. The hold-down anchors are used to limit global overturning under lateral loading by providing additional connection between the sill and the perimeter stud framing members.

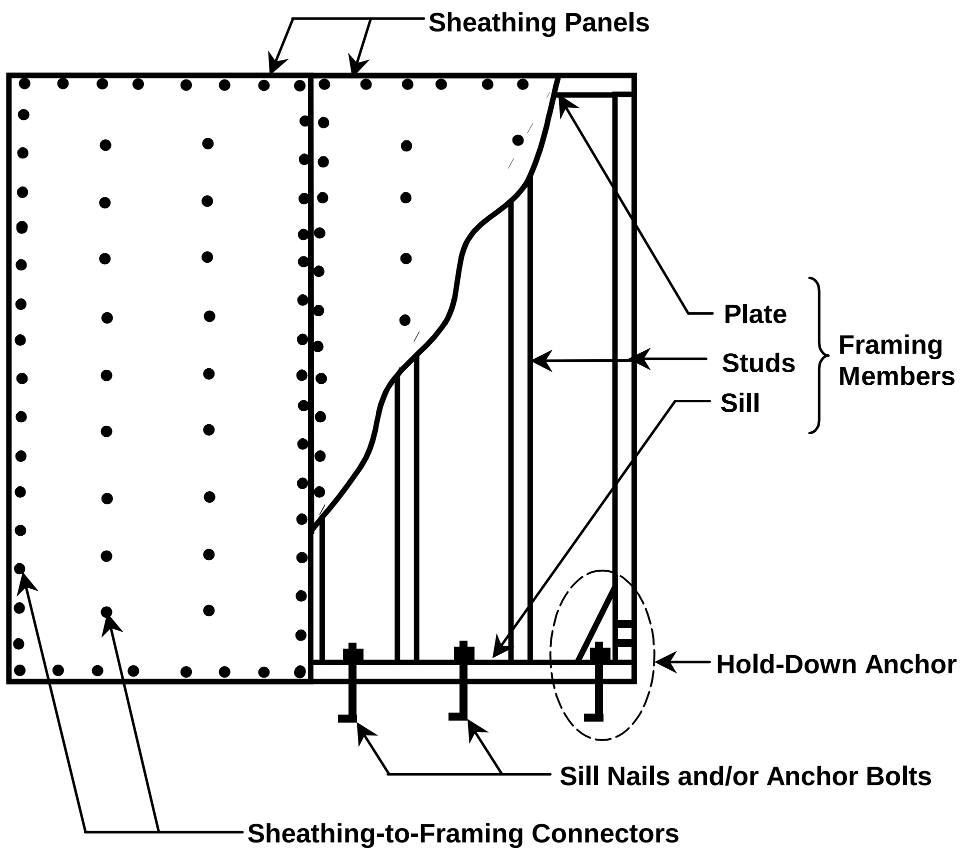


Figure 1. Components of a Typical Shear Wall.

1-2.2. Kinematic Assumptions

Figure 2 shows the deformed configuration (racking mode) of a typical shear wall under the action of a prescribed lateral force F_F or displacement U_F applied at the top of the wall.

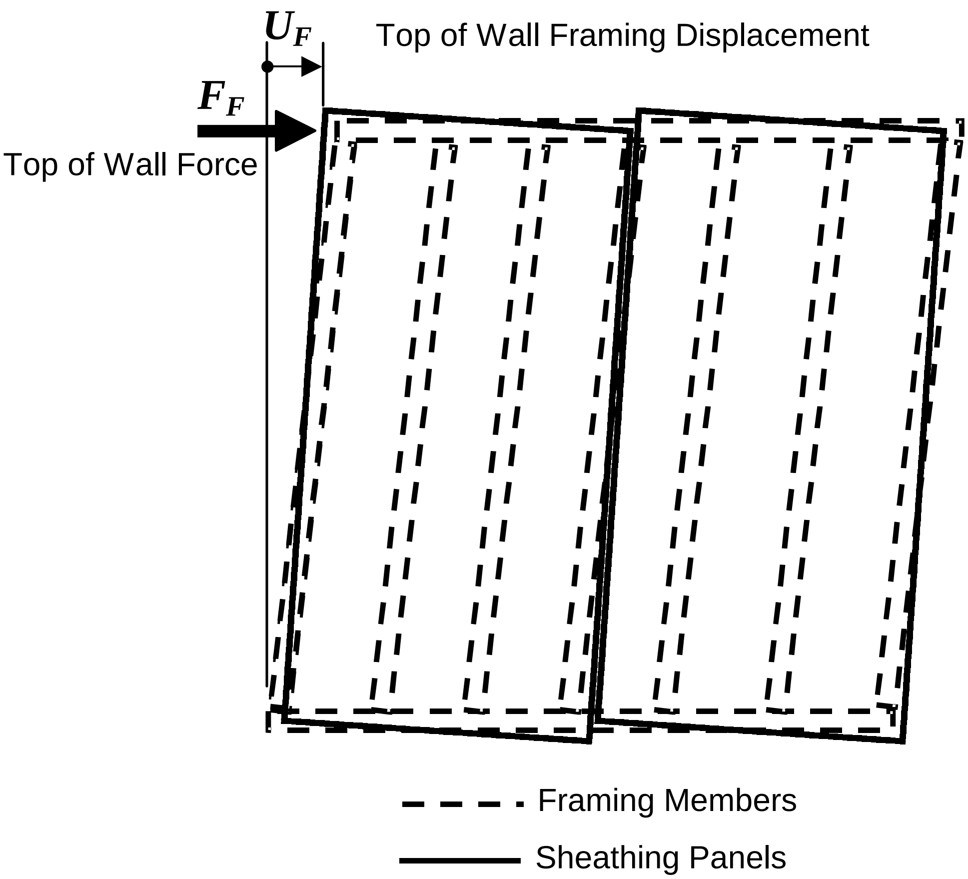


Figure 2. Racking Deformation of a Typical Shear Wall.

Under this action, the original orthogonal grid work of framing members distorts into a parallelogram with the top plate and sill remaining essentially horizontal. It is assumed herein that the sill is sufficiently anchored so that uplift is effectively eliminated. Previous research has shown that the in-plane bending of framing members has a second-order effect on wall response (Gupta and Kuo 1985). Hence, in this study the framing members are assumed to be rigid with pin-ended connections, given the nominal attachment that is prescribed for these members. As a consequence of these assumptions, the grid work of framing members alone is modeled as a mechanism with no lateral stiffness. As seen from Fig. 2, the lateral displacement of the framing can be characterized by a single degree-of-freedom; the top of frame lateral displacement or drift U_F.

As illustrated in Fig. 3 when the wall racks, each rectangular sheathing panel develops a uniform in-plane shear deformation U_s, superimposed on rigid-body translations $\overline{U}$ and $\overline{V}$ and rotation Θ. These rigid body modes are each measured with respect to the centroid of the panel $(\bar{x}, \bar{y})$ in the undeformed configuration. It is assumed that the sheathing panels have sufficient stiffness so that out of plane panel deformations can be ignored. Previous research supports this assumption for typical sheathed shear walls (Dolan 1989). Thus, each sheathing panel within the wall is assigned only 4 degrees of freedom: U_s, $\overline{U}$, $\overline{V}$ and Θ.

At the ultimate lateral load carrying capacity of a typical wood shear wall, the corresponding lateral drift is of the order of 2% to 4%. Consequently, in this study it is assumed that the individual deformations of the framing members and sheathing panels are relatively small. It then follows that a generic point Q on the sheathing panel with initial local panel coordinates (x,y) as shown in Fig. 3 experiences the following linearized deformations u_p and v_p under racking of the wall:

$$u_p = \overline{U} + 2\left(\frac{y}{h}\right)U_s - y\Theta \tag{1}$$

$$v_p = \overline{V} + x\Theta \tag{2}$$

with h the height of the panel under consideration. Corresponding to these panel displacements at the same initial point on the framing, the resulting linearized deformations are

$$u_f = \left(\frac{y + \bar{y}}{H} \right) U_F \tag{3}$$

$$v_f = 0 \tag{4}$$

with H the overall height of the wall. The above displacement field for the sheathing and framing can be written more succinctly in terms of the global displacement vector $D^T = [U_F, \quad U_s, \quad \bar{U}, \quad \bar{V}, \quad \Theta]$:

$$\begin{Bmatrix} u_p \\ v_p \\ u_f \\ v_f \end{Bmatrix} = \begin{bmatrix} 0 & 2y/h & 1 & 0 & -y \\ 0 & 0 & 0 & 1 & x \\ (y+\bar{y})/H & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 \end{bmatrix} D \tag{5}$$

Extension of the above presentation to walls with multiple sheathing panels is straightforward. For the general case of a wall with N_p sheathing panels, the number of degrees-of-freedom (NDOF) required to characterize the racking deformation of the shear wall is $NDOF = 4N_p + 1$.

The relative displacement between the sheathing and the framing induces deformations in the sheathing-to-framing connectors within the wall. In Fig. 3, this deformation is identified by δ for a generic connector fastened at one end to the framing member at point Q' and at the other end to the sheathing panel at point Q". Evaluation of δ is given by:

$$\delta = \sqrt{\delta_u^{\,2} + \delta_v^{\,2}} = \sqrt{(u_p - u_f)^2 + (v_p - v_f)^2} \tag{6}$$

As presented by Eq. (6), the connector deformation can be decomposed into a horizontal component δ_u and a vertical component δ_v, which, in turn, can be determined from the global degrees-of-freedom for the wall through Eq. (5).

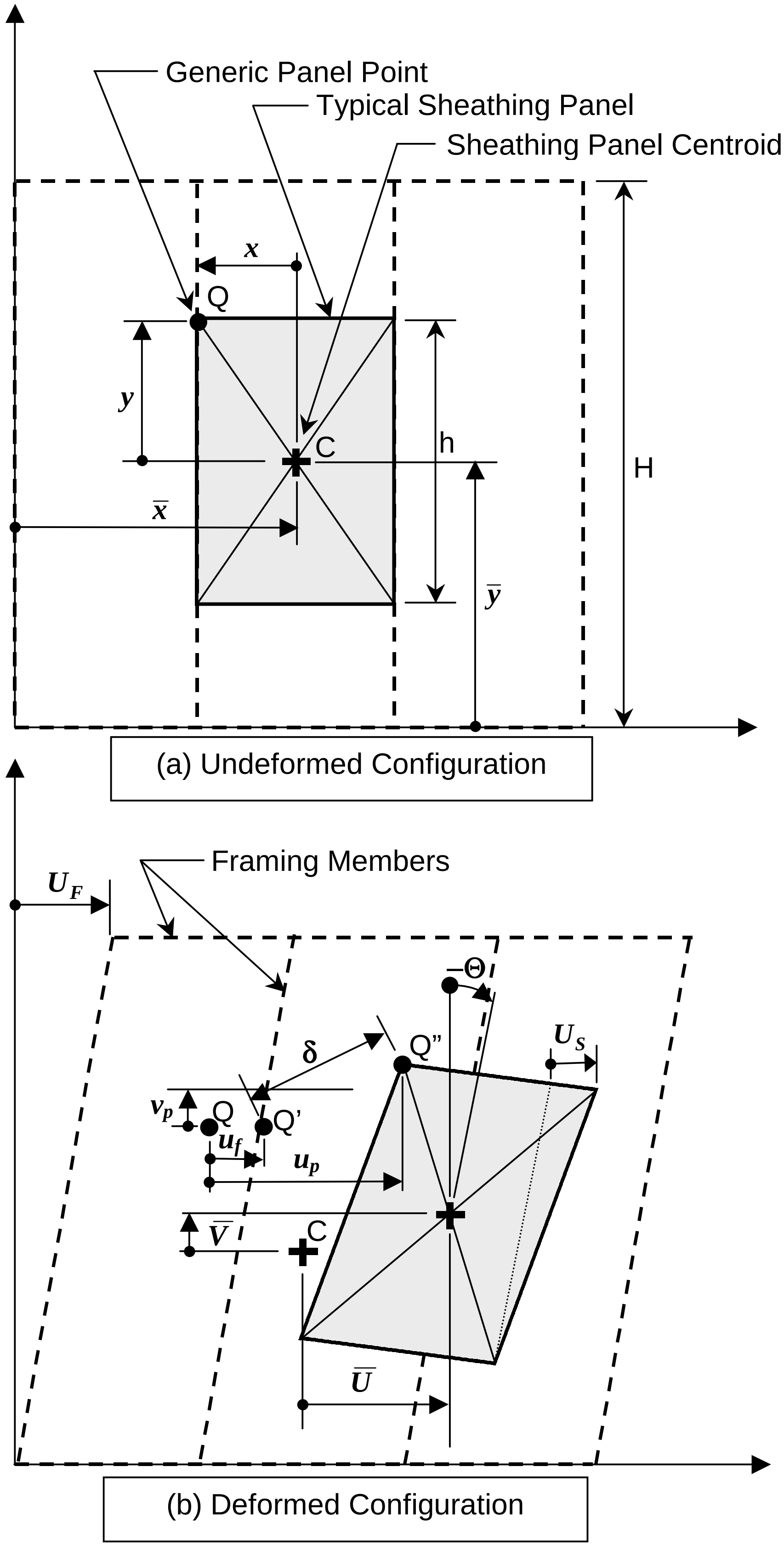


Figure 3. General Deformation of a Sheathing Panel within a Shear Wall.

Note that the kinematic assumptions outlined above for this shear wall model are essentially the same as those used previously by the second author in an earlier study (Filiatrault 1990).

1-2.3. Hysteretic Model of Sheathing-to-Framing Connectors

The load-deformation response of a dowel-type connector in a wood shear wall is highly non-linear under monotonic loading and exhibits pinched hysteretic behavior with strength and stiffness degradation under general cyclic loading (Dolan and Madsen 1992a). Each connector behaves essentially as an elasto-plastic pile (steel nail) embedded in a layered non-linear foundation (sheathing and framing material). Various researchers (Foschi 1974, 2000; Chui et al 1998) have used this structural analogy to develop fairly sophisticated finite element models for individual connectors. This approach is versatile and is capable of capturing the detailed cyclic response of a connector. However, it is computationally demanding to model each connector within a shear wall in this manner. A simpler and more efficient approach is to develop a specific hysteretic model based on a minimum number of path following rules that can reproduce the response of the connector under general cyclic loading. It is this latter method that is adopted for this study.

First, consider the response of a connector under monotonic loading. The non-linear load-deformation curve in Fig. 4, which was first proposed by Foschi (1977), is adopted for this study:

$$F = \begin{cases} \mathrm{sgn}(\delta) \cdot (F_0 + r_1 K_0 |\delta|) \cdot [1 - \exp(-K_0 |\delta| / F_0)], & |\delta| \le |\delta_u| \\ \mathrm{sgn}(\delta) \cdot F_u + r_2 K_0 [\delta - \mathrm{sgn}(\delta) \cdot \delta_u], & |\delta_u| < |\delta| \le |\delta_F| \\ 0, & |\delta| > |\delta_F| \end{cases} \tag{7}$$

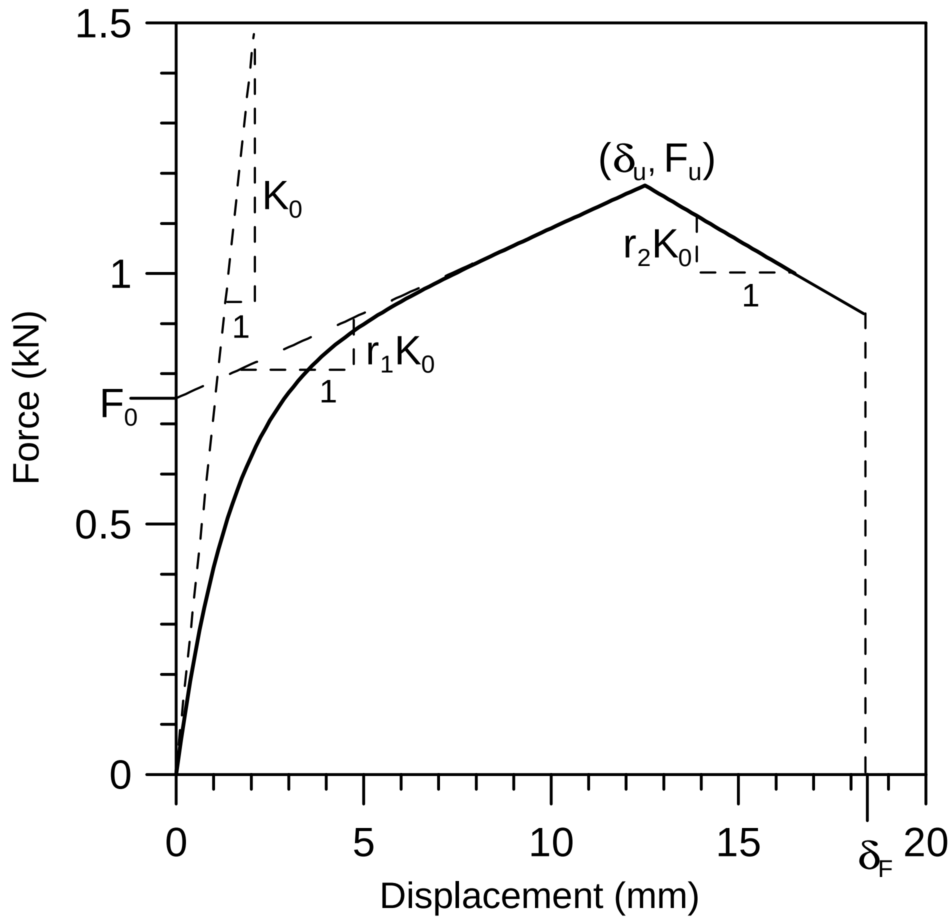


Figure 4. Force-Displacement Response of a Sheathing-to-Framing Connector under Monotonic Loading. (Hysteretic model fitted to connection test data for a 50 mm long spiral nail through a 9.5 mm thick OSB panel into a SPF No. 2 sawn lumber framing member (Durham 1998).

Application of this equation requires six physically identifiable parameters to be fitted to experimental data: F_0, K_0, r_1, r_2, δ_u and δ_F. Phenomenologically, Eq. (7) captures the crushing of the wood (framing and sheathing) along with yielding of the connector. Beyond the displacement δ_u, which corresponds to the ultimate load F_u, the load-carrying capacity is reduced because of connector withdrawal. Failure of the connector under monotonic load occurs at displacement δ_F.

Next, consider the load-deformation response of a connector under arbitrary cyclic loading as shown in Fig. 5. The basic path following rules, which define the proposed hysteretic model are identified and briefly discussed.

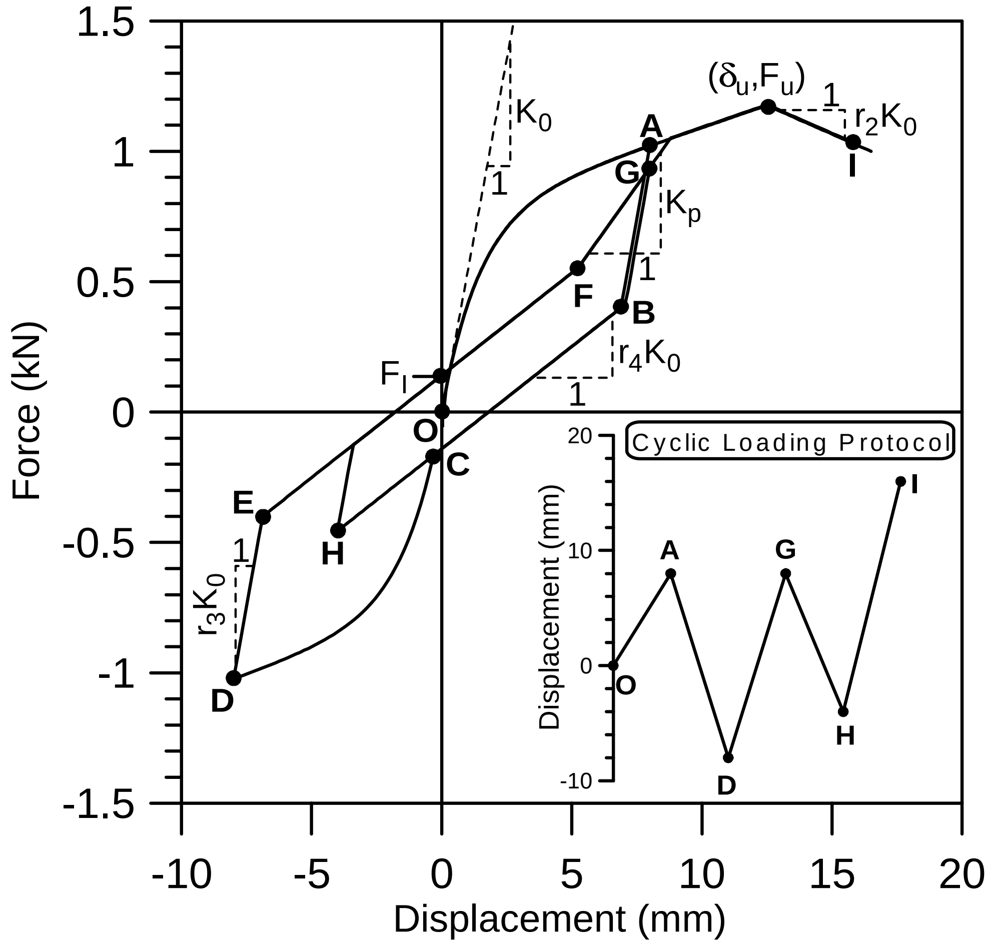


Figure 5. Force-Displacement Response of a Sheathing-to-Framing Connector Under Cyclic Loading. Hysteretic model fitted to connection test data for a 50 mm long spiral nail through a 9.5 mm thick OSB panel into a SPF No. 2 sawn lumber framing member (Durham 1998).

In Fig. 5 load-displacement paths OA and CD follow the monotonic envelope curve as expressed by Eq. (7). All other paths are assumed to exhibit a linear relationship between force and deformation. Unloading off the envelope curve follows a path such as AB with stiffness r_3K_0. Here, both the connector and wood are unloading elastically. Under continued unloading the response moves onto path BC, which has reduced stiffness r_4K_0. Along this path, the connector loses partial contact with the surrounding wood because of permanent deformation that was produced by previous loading, along path OA in this case. The slack response along this path characterizes the pinched hysteresis displayed by dowel connections under cyclic loading. 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

$$K_p = K_0\left(\frac{\delta_0}{\delta_{\max}}\right)^{\alpha} \tag{8}$$

with $\delta_0 = (F_0/K_0)$ and α a hysteretic model parameter which determine the degree of stiffness degradation. Note in Eq. (8) that K_p is a function of the previous loading history through the last unloading displacement δ_{un} off the envelope curve (corresponding to point A in Fig. 5), so that

$$\delta_{\max} = \beta\delta_{un} \tag{9}$$

where β is another hysteretic model parameter. The parameters α and β are obtained by fitting the model to connection test data. A consequence of this stiffness degradation is that it also produces strength degradation in the response. If on another cycle, the connector is displaced to δ_{un}, then the corresponding force will be less than F_{un} that was previously achieved. This strength degradation is shown in Fig. 5 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 stabilizes. This behavior is close to what has been observed in connector tests (Dolan and Madsen 1992a), unless nail fatigue becomes a factor.

1-2.4. Governing Equations

The equilibrium equations for the racking of a wood shear wall are obtained through application of the principle of virtual displacements (PVD):

$$\delta W_I = \delta\left(W_F + W_s + W_C\right)_I = \delta(W_S + W_C)_I = \delta W_E \tag{10}$$

PVD

with the internal work W_I comprised of contributions from the framing W_F, sheathing W_S and connectors W_C and the external work W_E arising from the applied racking load. As discussed previously, it is assumed in this model that the framing alone is a mechanism of pin-connected rigid members; hence the internal work associated with this component of the wall is zero.

The internal work absorbed during the uniform shear deformation U_s of a sheathing panel is given by

$$W_S = \frac{1}{2}\int_{V_p} \tau\gamma dV = \left(\frac{2Gbt_p}{h}\right)U_S^2 = D^T B_S \left(\frac{2Gbt_p}{h}\right) B_S D \tag{11}$$

where b, h, t_p and V_p are, respectively, the width, height, thickness and volume of the panel under consideration. In the formulation of Eq. (11) it is assumed the shear stress τ that develops in the panel is linearly related to the strain field γ through the panel's shear modulus G. The final expression in Eq. (11) allows W_S to be expressed in terms of the global displacement vector D, by setting $B_S = [0, \quad 1, \quad 0, \quad 0, \quad 0]$.

The energy absorbed by all N_c sheathing-to-framing connectors within a given sheathing panel is given by

$$W_C = \sum_{j=1}^{N_C}\left(\int_0^{\delta_j} F_j d\xi\right) \tag{12}$$

where $\boldsymbol{F_j}$ is the connector force resulting from a relative deformation of $\boldsymbol{\delta_j}$ between the sheathing and the framing. Determination of $\boldsymbol{F_j}$, for a given $\boldsymbol{\delta_j}$, is obtained from the rules that define the hysteretic model presented previously. Using Eqs. (5) and (6), Eq. (12) can be expressed in terms of the global displacement vector $\boldsymbol{D}$.

It has been observed that under monotonic loading of a shear wall the deformation trajectory of each connector is essentially unidirectional (Tuomi and McCutcheon 1978). This behavior is illustrated in Fig. 6a, with the connector modeled by a single non-linear spring element.

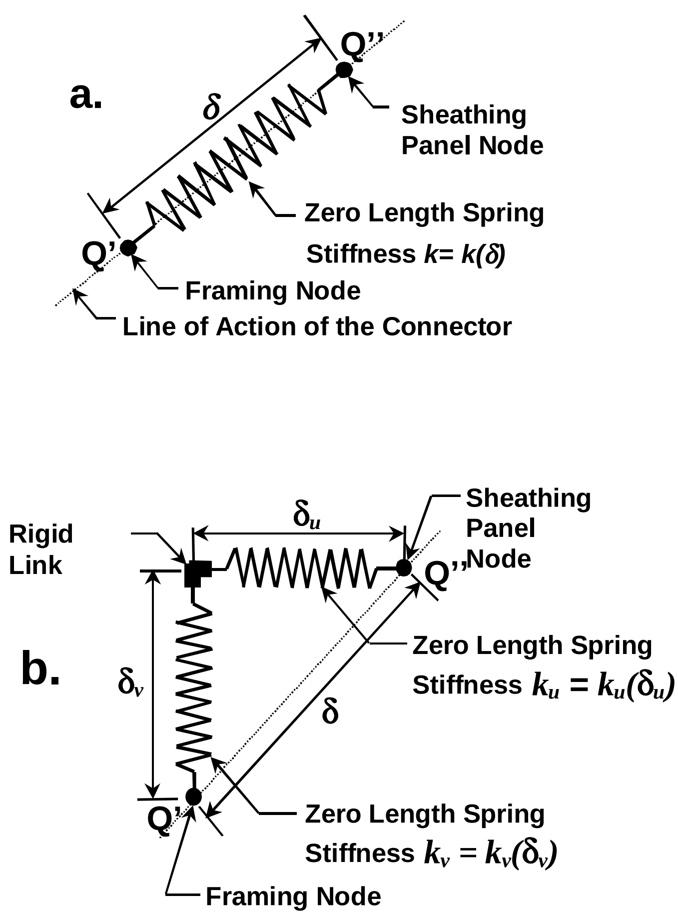


Figure 6. Sheathing-to-framing Connector Elements:
a. Monotonic Loading, b. Cyclic Loading.

Under general cyclic loading of the wall, the displacement trajectory of a connector can be bi-directional. This added complexity in the wall response does not allow for a straightforward evaluation of Eq. (12) in terms of the global degrees-of-freedom. To facilitate a solution, each sheathing-to-framing connector is modeled using two uncoupled orthogonally oriented non-linear spring elements, as shown in Fig. 6b. This modeling approach has been utilized in many of the existing shear wall models discussed previously (Itani and Cheung 1984; Gupta and Kuo, 1985; Dolan 1989 and White and Dolan 1995), even for the case of monotonic loading where it is not actually required. It is important to note that under an imposed deformation, as given by Eq. (6), the use of two orthogonal uncoupled springs is only structurally equivalent, in terms of resultant force and stiffness, to one spring if each spring is linear elastic with the same stiffness. For the hysteretic model of the sheathing-to-framing connector presented herein, the use of two uncoupled nonlinear springs will generally over-estimate the force and stiffness developed by the connector. A method to adjust for the resulting stiffer wall response will be outlined in a subsequent section.

For racking of a shear wall, the external work is simply the product of the applied top of wall framing displacement $\boldsymbol{U_F}$ and the induced top of wall lateral load $\boldsymbol{F_F}$. It then follows that the right hand side of Eq. (10) is given by

$$\boldsymbol{\delta W_E} = \boldsymbol{F_F} \cdot \boldsymbol{\delta U_F} = \boldsymbol{\delta D^T F} \tag{13}$$

where $\boldsymbol{F} = [\boldsymbol{F_F}, \quad \boldsymbol{0}, \quad \boldsymbol{0}, \quad \boldsymbol{0}, \quad \boldsymbol{0}]^{\boldsymbol{T}}$ is the global force vector.

Substitution of Eqs. (11) to (13) into Eq. (10) yields the non-linear governing equilibrium equations for the racking response of the shear wall assembly:

$$\boldsymbol{K_S D} = \boldsymbol{F} \tag{14}$$

where $\boldsymbol{K_S = K_S(D)}$ is the global secant stiffness matrix which is a function of the displacement vector $\boldsymbol{D}$. The explicit evaluation of $\boldsymbol{K_S}$ is given in Appendix A. The formulation given above for obtaining the non-linear equilibrium equations has been, for simplicity and clarity, presented for a shear wall with only one sheathing panel. For this special case the model has only 5 degrees-of-freedom. Extension to a wall with $\boldsymbol{N_p}$ sheathing panels is straightforward and leads to a model with $\boldsymbol{4N_p+1}$ degrees-of-freedom to be solved.

1-2.5. Displacement Control Solution Strategy

The equilibrium equations, given by Eq. (14), are solved using an incremental-iterative displacement control solution strategy (Batoz and Dhatt 1979; Ramm 1981). Suppose that the configuration of the wall is known at load-step $\boldsymbol{t}$ and the solution is sought at $\boldsymbol{t+\Delta t}$. The incremental equilibrium equations can be written as

$$^{(t+\Delta t)}\boldsymbol{K_T}\,^{(t+\Delta t)}\boldsymbol{\Delta D} = {}^{(t+\Delta t)}\Delta\lambda \boldsymbol{F_o} + {}^{(t)}\boldsymbol{R} \tag{15}$$

with

$$^{(t)}\boldsymbol{R} = {}^{(t)}\boldsymbol{F} - {}^{(t)}\boldsymbol{K_S}\,^{(t)}\boldsymbol{D} \tag{16}$$

In Eq. (15), $\boldsymbol{K_T}$ is the global tangent stiffness matrix, λ is the load factor applied to the reference global load vector $\boldsymbol{F_o}$ and $\boldsymbol{R}$ is the global residual force vector, which is rewritten in Eq. (16) in terms of known quantities. To solve Eq. (15), the incremental global displacement vector is decomposed into two parts:

$$^{(t+\Delta t)}\boldsymbol{\Delta D} = {}^{(t+\Delta t)}\Delta\lambda \boldsymbol{\Delta D^I} + {}^{(t)}\boldsymbol{\Delta D^{II}} \tag{17}$$

which, in turn, produces two systems of equations to be solved:

$$\begin{aligned} {}^{(t+\Delta t)}\boldsymbol{K_T}\,{}^{(t+\Delta t)}\Delta\boldsymbol{D}^{I} &= \boldsymbol{F_o} \\ {}^{(t+\Delta t)}\boldsymbol{K_T}\,{}^{(t+\Delta t)}\Delta\boldsymbol{D}^{II} &= {}^{(t)}\boldsymbol{R} \end{aligned} \tag{18}$$

Under displacement control, the top of wall translation of the shear wall $\boldsymbol{U_F} \equiv \boldsymbol{D_1}$ is prescribed. Hence, from Eq. (17) the following constraint equation is obtained:

$${}^{(t+\Delta t)}\Delta\boldsymbol{D_1} = {}^{(t+\Delta t)}\Delta\lambda\,{}^{(t+\Delta t)}\Delta\boldsymbol{D_1}^{I} + {}^{(t+\Delta t)}\Delta\boldsymbol{D_1}^{II} = \boldsymbol{0} \tag{19}$$

from which the increment in the load factor can be determined:

$${}^{(t+\Delta t)}\Delta\lambda = -\frac{{}^{(t+\Delta t)}\Delta\boldsymbol{D_1^{II}}}{{}^{(t+\Delta t)}\Delta\boldsymbol{D_1^{I}}} \tag{20}$$

For a given displacement increment, Newton-Raphson iterations are performed on Eq. (18) until the new equilibrium configuration of the shear wall is obtained to within a specified tolerance on the residual force vector. The total displacements and forces acting on the shear wall are then updated from the previous increment:

$$\begin{aligned} {}^{(t+\Delta t)}\boldsymbol{D} &= {}^{(t)}\boldsymbol{D} + \left[{}^{(\Delta t)}\Delta\boldsymbol{D}^{I} + {}^{(\Delta t)}\Delta\boldsymbol{D}^{II}\right] \\ {}^{(t+\Delta t)}\boldsymbol{F} &= {}^{(t+\Delta t)}\lambda\boldsymbol{F_o} \end{aligned} \tag{21}$$

The above presented solution strategy fails when the global tangent stiffness matrix $\boldsymbol{K_T}$ is non-positive definite. To overcome this limitation an artificial lateral linear spring (Ramm 1981) is introduced at the top of the shear wall so that the combined tangent stiffness matrix of the spring plus shear wall remains positive definite over the cyclic loading protocol. This can be achieved by setting the axial stiffness of the artificial spring equal to the initial stiffness of the shear wall. The force developed by the spring under the prescribed displacement $\boldsymbol{U_F}$ is removed at the end of each loading step to obtain the top of wall force in the shear wall from Eq. (21).

It is well known that the hysteretic response of a shear wall is largely determined by the hysteretic characteristics of the sheathing-to-framing connectors. In the model presented herein, the connector properties are defined in terms of a finite set of path following rules. In order for the load-displacement response of the individual connectors to be captured, according to these rules, the step size in the displacement protocol for the overall wall must not be too large. Unfortunately, this limiting step size is not known a priori. To advance the solution, an adaptive strategy has been implemented with a variable step size determined by a bisectional search to ensure convergence at each step of the specified loading protocol.

The modeling features described herein for the cyclic response of wood shear walls have been incorporated in the computer program CASHEW: Cyclic Analysis of SHEar Walls.

1-2.6. Idealization of a Shear Wall as a Single Degree-of-Freedom System

The CASHEW model presented herein is capable of predicting the load-displacement response of wood shear walls under cyclic loading. To perform this analysis for a given shear wall configuration only the shear modulus of the sheathing panels and cyclic test data from the sheathing-to-framing connectors are required as input data.

It is obvious that more information can be obtained about the structural performance of a shear wall through performing a cyclic analysis compared to just a monotonic pushover analysis. It is equally obvious that the dynamic analysis of a shear wall provides still greater information, especially as it relates to the seismic design problem. In a number of previous studies, the dynamic behavior of shear walls has been investigated using non-linear single degree-of-freedom (SDOF) models (*e.g.* Stewart 1987 and Foliente 1995). In these studies, hysteretic elements, which modeled the global wall behavior under cyclic loading, were fitted to experimental data obtained from full-scale shear walls tests. These hysteretic elements were incorporated into non-linear SDOF dynamic analysis programs to numerically predict the seismic response of shear walls. This research work has shown that acceptable results can be obtained through a SDOF idealization of a wood shear wall. These previously cited studies, however, were limited by the need of full-scale cyclic test data to calibrate the hysteretic shear wall elements. The same procedure can be adopted herein with the exception that the CASHEW model can be used to calibrate the SDOF hysteretic model for the dynamic analysis. In this way the dependency on full-scale shear wall test data is eliminated.

It is well known that the hysteretic response of a typical shear wall has the same defining characteristics (pinched behavior, strength and stiffness degradation, *etc*.) as those exhibited by an individual sheathing-to-framing connector under cyclic loading (Dolan and Madsen 1992b.) Consequently, the hysteretic model presented earlier, which was applied to sheathing-to-framing connectors, can be used to represent the global response of the shear wall under cyclic loading with appropriately applied parameter values. These hysteretic model parameters for the wall can be identified through a non-linear functional minimization procedure. The objective function to be minimized is the cumulative error between the restoring force $\boldsymbol{F_F}$ developed in the shear wall as predicted by the SDOF hysteretic model and that obtained from the CASHEW model when subjected to the same cyclic loading protocol.

As an option within the CASHEW computer program, the system identification procedure presented above is utilized to obtain the defining parameters of an equivalent SDOF hysteretic shear element once the cyclic analysis of the wall has been completed. This system identification procedure is applied exclusively to the CUREe-Caltech testing protocol (Krawinkler et al., 2000) that has been recently developed under the CUREe-Caltech Woodframe Project. This protocol, which can be used automatically in the CASHEW program, is discussed in Section 1-4.

1-3. MODEL VERIFICATION

In this section, the predictive capabilities of the CASHEW model are compared with experimental results from full-scale monotonic and cyclic shear wall tests performed at the University of British Columbia under a separate investigation (Durham 1998; Durham et al. 1999).

In this experimental study, the dimensions of the test shear walls were 2.4 m x 2.4 m. All framing material was 38 mm x 89 mm dimensional lumber. The top plate and end studs consisted of double members, while the sole plate and the interior studs were single members. Studs were spaced at 400 mm on center. Conventional corner hold-downs were used to prevent overturning of the wall and to ensure a racking mode of deformation. The sheathing panels were 9.5 mm thick oriented strand board (OSB), with an assigned elastic shear modulus of 1.5 GPa. Three panels were used to sheath the wall: a 1.2 m x 2.4 m panel covered the bottom half of the wall and two 1.2 m x1.2 m panels covered the top half of the wall. Horizontal blocking was used at mid-height along the wall between the sheathing panels. The sheathing-to-framing connectors were pneumatically driven 50 mm long spiral nails with a shank diameter of 2.67 mm. Nails were spaced at 150 mm on center along all panel edges and 300 mm spacing for all interior studs. This testing program also collected sheathing-to-framing connector data and fitted it to a hysteretic connector model similar to the one presented in this study. Table 1 lists the computed sheathing-to-framing connector parameters obtained from the experimental study (Durham 1998).

Figures 4 and 5 show, respectively, the monotonic and cyclic load-displacement behavior of the sheathing-to-framing connectors when the hysteretic connector model is fitted with these parameters.

Table 1. Sheathing-to-Framing Connector Parameters for 50 mm Long Spiral Nails (from Durham 1998)

K_0 (kN/mm)	r_1	r_2	r_3	r_4	F_0 (kN)	F_I (kN)	Δ_u (mm)	α	β
0.561	0.061	-0.078	1.40	0.143	0.751	0.141	12.5	0.8	1.1

Figure 7 illustrates the cyclic loading protocol used in this test program. As is standard practice in shear wall testing, this cyclic loading protocol was scaled by the wall's load-displacement performance under monotonic loading. With this protocol, the wall is first subjected to 3 cycles with a maximum drift equal to the displacement corresponding to 50% of the wall's ultimate load established under monotonic testing. This displacement level is denoted as $\Delta_{0.5Pu}$ in Fig. 7. The protocol then has 3 cycles at $\Delta_{0.8Pu}$, followed by one trailing cycle back at $\Delta_{0.5Pu}$ and finishing with a uni-directional push-over of the wall until failure.

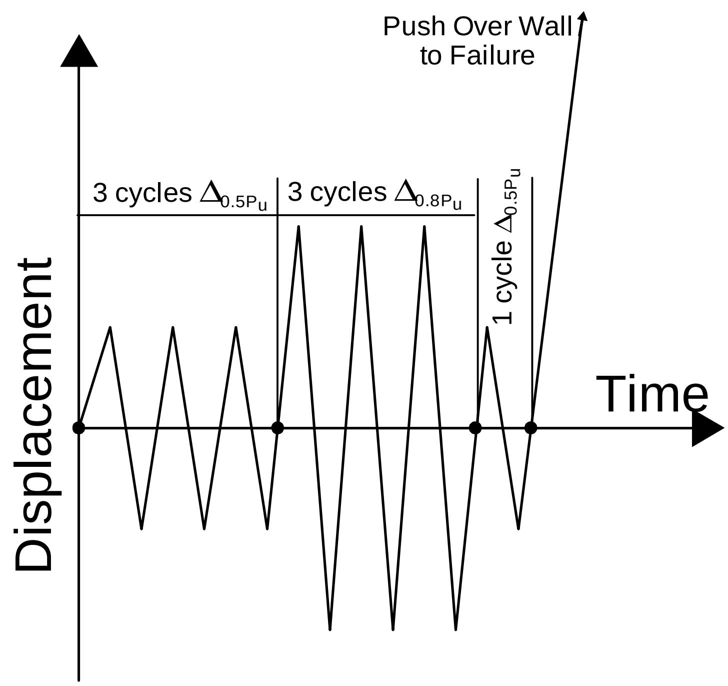


Figure 7. Cyclic Loading Protocol for Shear Wall Tests.

The experimentally obtained load-displacement response of the shear wall under both monotonic loading and cyclic loading is presented in Fig. 8. The corresponding predictions by the numerical model, through the program CASHEW, are given in Fig. 9. A visual comparison of Figs. 8 and 9 shows that good agreement is achieved between the model prediction and the experimental result for the prescribed cyclic loading. In particular, the observed stiffness and strength degradation that the test wall exhibited under the loading protocol was fully captured by the numerical model. However, the model did not perform as well in predicting the response during the single trailing cycle of the loading protocol. Key results from these two figures are the lateral load carrying capacity P_u of the wall and the corresponding drift Δ_u, which are summarized in Table 2. Under cyclic loading the difference between the test results and the model predictions are 7.8% and 9.1%, respectively, for the ultimate load and displacement.

Table 2. Summary of Test Results and Model Predictions under Monotonic and Cyclic Loading.

	Monotonic Loading		Cyclic Loading		
	P_u (kN)	Δ_u (mm)	P_u (kN)	Δ_u (mm)	E_a (kN-m))
Test	17.4	57.4	20.4	66.0	2.59
Model	22.0	60.0	22.0	60.0	2.68
% Difference	26.4	4.5	7.8	9.1	3.5

Another means of verifying the accuracy of the model is through the evaluation of the energy absorbed by the wall under the cyclic loading protocol:

$$\boldsymbol{E_a = \int_0^{\Delta_F} F_T \Delta \, d\Delta} \tag{22}$$

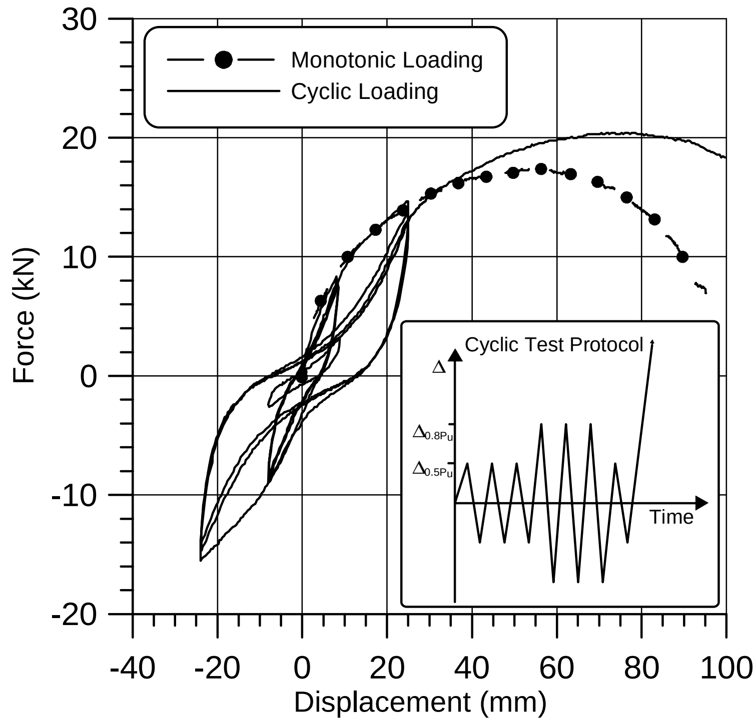


Figure 8. Experimental Monotonic and Cyclic Shear Wall Tests.

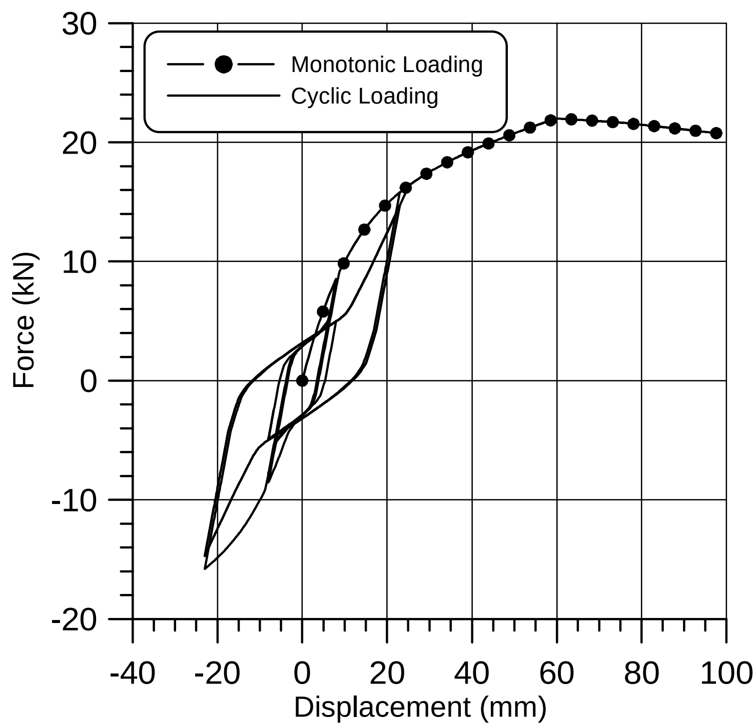


Figure 9. CASHEW Predictions of Monotonic and Cyclic Shear Wall Tests.

In the evaluation of Eq. (22), which must be performed numerically, the limits of integration track the full displacement of the wall under the loading protocol. The comparison of energy absorbed during the cyclic test and the model's prediction is presented in Fig. 10, which reveals very good agreement. Numerically, the difference in the total energy absorbed between the test result and the model prediction is only 3.5%.

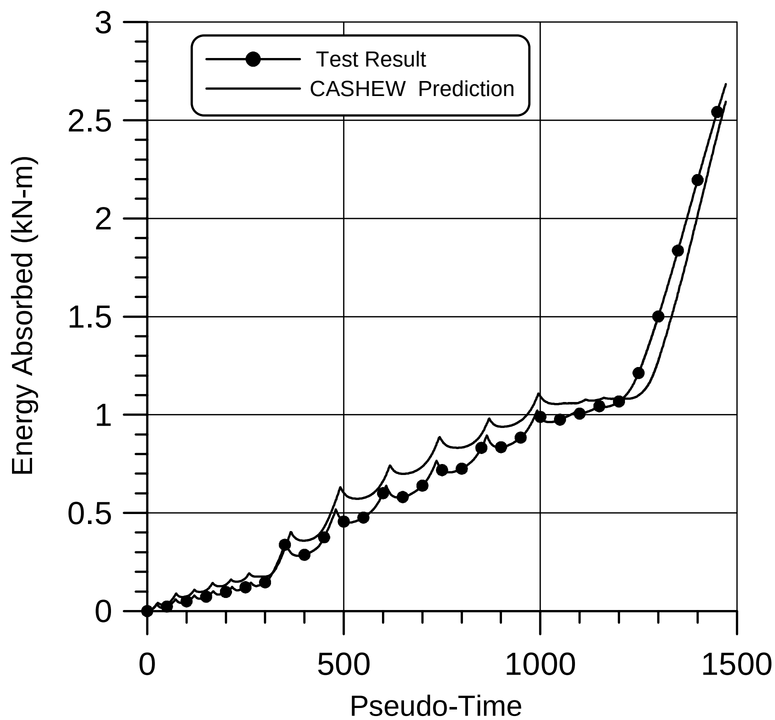


Figure 10. Energy Absorbed during the Cyclic Shear Wall Test: Experimental Result and Cashew Prediction

Comparing the test result and the model prediction of the load-displacement response of the wall under monotonic loading, as presented by Figs. 8 and 9 and as summarized in Table 2, reveals a fairly significant discrepancy, with the estimate of ultimate load differing by 26.4%. This difference may be attributable to the variability in construction quality between the two test walls. Durham noted that with the wall tested monotonically there was "observed poor nailing" (Durham 1998). This example reinforces the point that a purely deterministic evaluation should not be made between a single test result and a model's prediction of that test. It is to be expected that similarly constructed shear walls will exhibit variability in their response under load. Unfortunately, the quantification of this variability has not been a primary objective of testing programs investigating shear wall behavior.

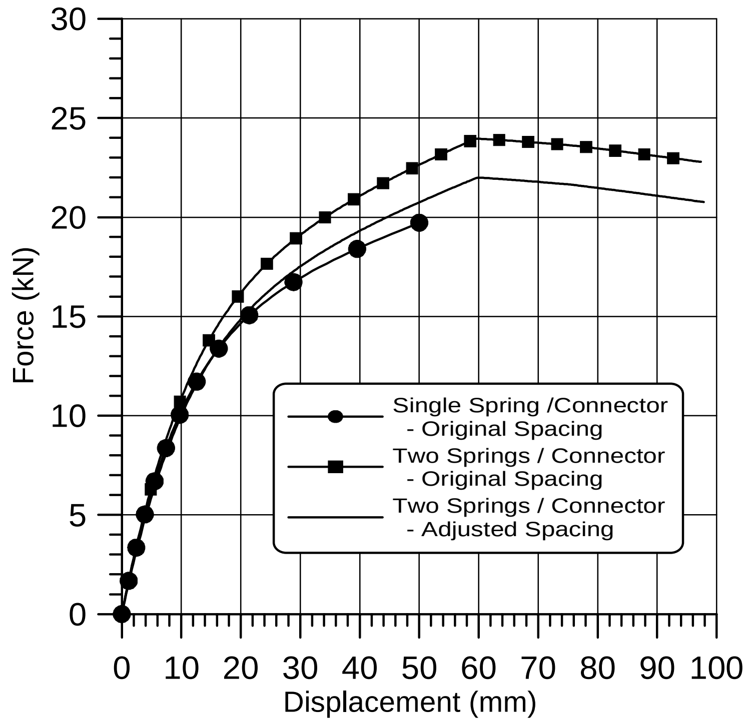


Figure 11. Monotonic Load-Displacement Response using One and Two Spring Sheathing-to Framing Connector Models in CASHEW.

In the model verification study presented above, each sheathing-to-framing connector was represented by two orthogonal uncoupled non-linear springs as discussed previously. However, the specified connector spacing was adjusted so that the monotonic load-displacement response agreed, in terms of energy absorbed by the wall up to a prescribed drift level, with the prediction based on using only one non-linear spring per connector with the spacing unchanged. As shown in Fig. 11, if this adjustment is not made the model over predicts the initial wall stiffness and the ultimate load carrying capacity. Surprisingly, this result has not been discussed in other research work, which has used two uncoupled non-linear springs to model each connector.

1-4. THE CUREe-CALTECH TESTING PROTOCOL

The CASHEW program can be run under any displacement protocol at the top of the wall specified by the user. As an option, the CUREe-Caltech Woodframe Project protocol for deformation controlled quasi-static cyclic loading (Krawinkler et al., 2000) can be selected automatically. The primary objective in developing this protocol is to evaluate capacity level seismic performance of components of woodframe structures subjected to ordinary (not near-fault) ground motions whose probability of exceedance in 50 years is 10 %. The development of the loading protocol is based on the results of non-linear dynamic analysis of representative SDOF hysteretic systems subjected to ordinary ground motions. The chosen ground motions are specific to California conditions with particular weighting to the Los Angeles area. Cumulative damage concepts were employed to transform the time history responses into a representative deformation controlled loading history. The protocol includes deformation cycles due to smaller events prior to the capacity level event. As formulated, this protocol may not be applicable to limit states other than capacity.

As used by the CASHEW model, this loading history follows the pattern given in Fig. 12. This protocol is an abbreviation of the ***basic loading history*** specified by Krawinkler et al. (2000), with the initial cycles below 0.20Δ eliminated since they produce, in general, nearly elastic response in the shear wall. Also, the protocol in CASHEW terminates at 1.5Δ, at which point the wall is expected to have very little remaining load-carrying capacity.

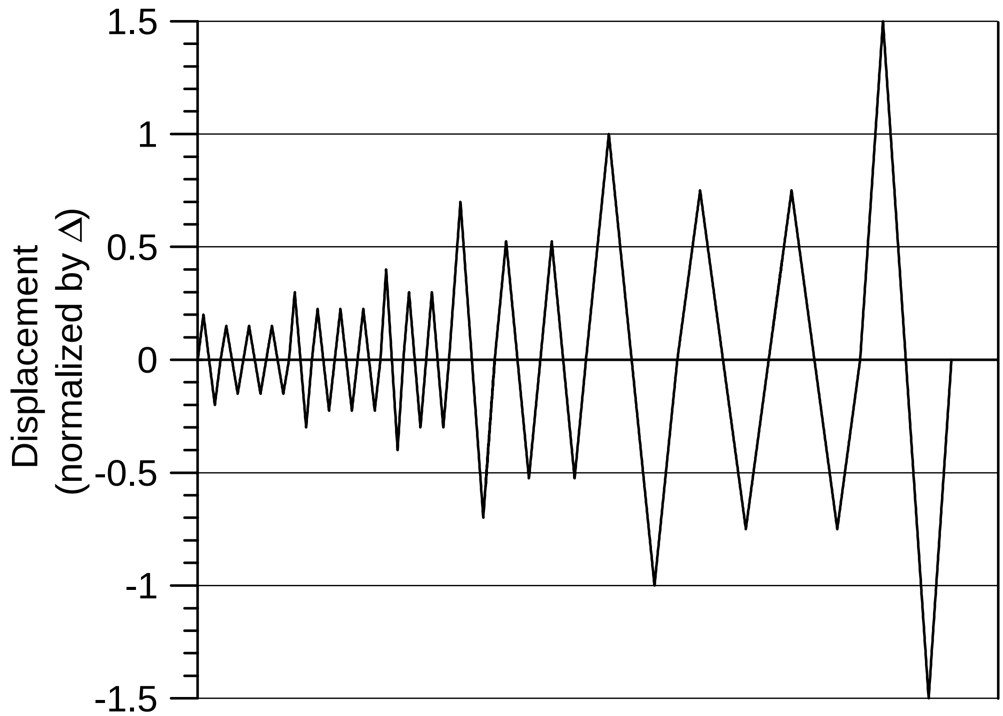


Figure 12. CUREe-Caltech Testing Protocol, as Implemented in the CASHEW Program.

This loading history is defined by variations in displacement amplitudes, scaled by the reference displacement Δ. The reference displacement, Δ, represents the drift capacity of the shear wall being analyzed. This reference displacement is estimated by CASHEW through the execution of an analysis under monotonic loading. This monotonic analysis provides a prediction on the monotonic displacement capacity, Δ_m. This capacity is defined as the displacement at which the applied load drops, for the first time, below 80% of the maximum load that was applied to the specimen. CASHEW considers that $\Delta = 0.6\Delta_m$. Once the reference displacement, Δ has been established, CASHEW scales the loading history shown in Fig. 12 accordingly and automatically performs the cyclic analysis of the shear wall considered. Note that the system identification procedure of an equivalent single degree-of-freedom described in Section 1-2.4 is only executed when the CUREe-Caltech Testing protocol is specified.

As an example of shear wall behavior under the CUREe-Caltech Woodframe Project testing protocol, the load-displacement response predicted by CASHEW for the test shear wall of Section 1-3 is presented in Fig. 13. This prediction of shear wall behavior was obtained using the sheathing-to-framing connector properties given in Table 1, except that parameter r_4 was reduced to a value of 0.05 (from 0.143). The particular test protocol used in the UBC test study did not include cyclic behavior near the ultimate capacity of the wall. This resulted in an unrealistically high value being assigned to r_4 compared to other studies (Dolan 1989). Examining Figs. 9 and 13 shows that the CUREe-Caltech protocol does not suffer this shortcoming. It is of interest to note that for this particular wall CASHEW predicts a displacement at ultimate load $\Delta_u = 60.0$ mm and the CUREe-Caltech protocol is scaled using a $\Delta = 59.0$ mm. The intent in the construction of the CUREe-Caltech protocol was to have $\Delta \approx \Delta_u$. For this particular shear wall this is the case.

Finally shown in Fig. 14 is the prediction of load-displacement response of the test shear wall under the CUREe-Caltech protocol when modeled as an equivalent SDOF hysteretic shear element with system parameters obtained by CASHEW. In comparing Figs. 13 and 14 it is observed that there is very good correlation between these two results. As expected the hysteretic response of the SDOF model is comprised of straight-line interior branches. For the full shear wall model response, shown in Fig. 13, there are smooth transition between these branches. This difference occurs because the load-displacement state of each sheathing-to-framing connector element in the model is contributing to the global wall response. This example supports the proposition that an appropriately formulated and fitted SDOF model can adequately represent the global cyclic racking response of a shear wall.

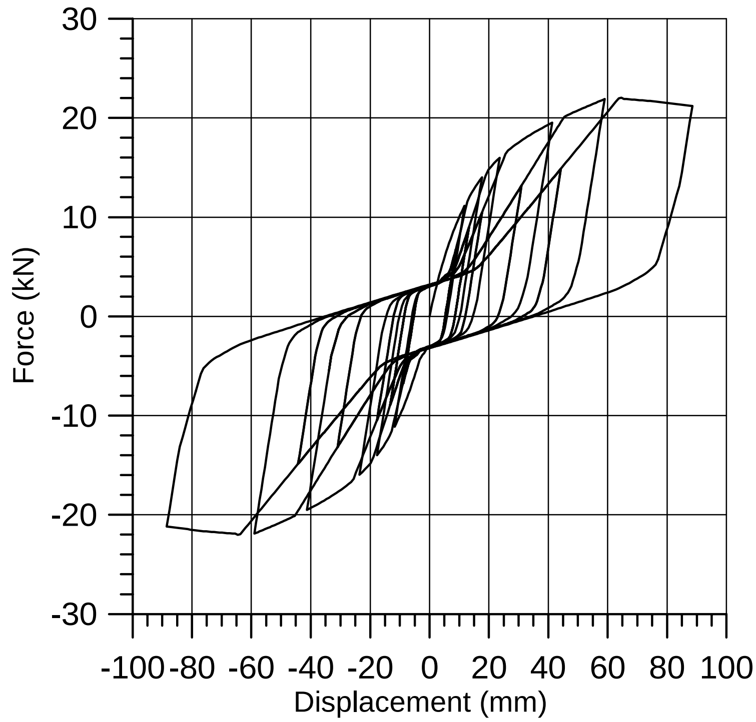


Figure 13. CASHEW Prediction of the Load-Displacement Response of Durham's Test Shear Wall Under the CUREe-Caltech Testing Protocol.

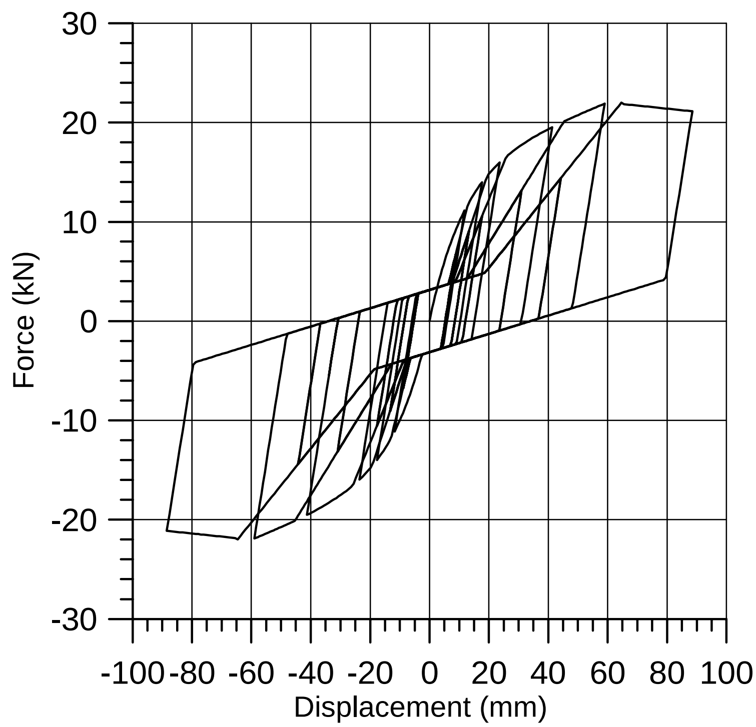


Figure 14. Equivalent SDOF Model Prediction of the Load-Displacement Response of Durham's Test Shear Wall Under the CUREe-Caltech Testing Protocol.

1-5. CONCLUSIONS

A simple numerical formulation for the structural analysis of wood framed shear walls under arbitrary cyclic loading has been elaborated based on the hysteretic properties of sheathing-to-framing connectors. The resulting numerical model, incorporated in the computer program CASHEW: Cyclic Analysis of SHEar Walls, is able to predict the load-displacement response and energy dissipation characteristics of wood shear walls, with or without opening, under arbitrary quasi-static cyclic loading. The model has been verified against full-scale tests of wood framed shear walls subjected to monotonic and cyclic loading. The predictions of the model agreed well with the experimental results. In addition, it has been shown that the cyclic load-displacement response of a shear wall can be well represented by an equivalent SDOF hysteretic shear element. The identification of the system parameters for this equivalent SDOF model can be obtained through the CASHEW program.

1-6. REFERENCES

Batoz, J.L. and Dhatt, G. (1979). "Incremental displacement algorithms for nonlinear problems." *Int. J. Numer. Methods in Engrg*, 14, 1262-1267.

CEN (1995). "Timber structures – test methods- cyclic testing of joints made with mechanical fasteners." *pr EN 12512*, European Committee for Standardization , Brussels, Belgium.

Chui, Y.H., Ni, C., and Jaing, L. (1998). "Finite-element model for nailed wood joints under reversed cyclic load." *J. Struct. Engrg, ASCE*, 124(1), 96-103.

CoLA/UCI Committee (1999). "Cyclic racking shear tests for light framed shear walls- August 1999." City of Los Angeles/ University of California, Irvine, CA.

Commins, A. and Gregg, R.C. (1994). "Effect of hold-downs and stud-frame systems on the cyclic behavior of wood shear walls." *Proc., Research Needs Workshop on Analysis, Design and Testing of Timber Structures Under Seismic Loads*, G.C. Foliente, ed., Forest Products Laboratory, University of California, Richmond, CA., 142-146.

Dolan J.D. and Foschi, R.O. (1991). "Structural analysis model for static loads on timber shear walls." *J. Struct. Engrg, ASCE*, 117(3), 851-861.

Dolan, J.D. (1989). "The dynamic response of timber shear walls." Ph.D. thesis, University of British Columbia, Vancouver, Canada.

Dolan J.D. and Madsen B. (1992a). "Monotonic and cyclic nail connection tests." *Can. J. of Civ. Engrg*., Ottawa, 19(1), 97-104.

Dolan J.D. and Madsen B. (1992b). "Monotonic and cyclic tests of timber shear walls." *Can. J. of Civ. Engrg*., Ottawa, 19(4), 415-422.

Durham, J.P. (1998). "Seismic response of wood shearwalls with oversized oriented strand board panels." M.A.Sc. thesis, University of British Columbia, Vancouver, Canada.

Durham J., Prion H.G.L., Lam, F., and He, M. (1999). "Earthquake resistance of shearwalls with oversize sheathing panels." *Proc., 8th Canadian Conf. on Earthquake Engrg*, Vancouver, Canada, 161-166.

Filiatrault, A. (1990). "Static and dynamic analysis of timber shear walls." *Can. J. of Civ. Engrg*., Ottawa, 17(4), 643-651.

Foliente G.C. (1995). "Hysteresis modeling of wood joints and structural systems." *J. Struct. Engrg, ASCE*, 121(6), 1013-1022.

Foschi, R.O. (1974). "Load-slip characteristics of nails." *Wood Sci*., 7(1), 69-74.

Foschi, R.O. (1977). “Analysis of Wood Diaphragms and Trusses, Part 1: diaphragms.” *Can. J. of Civ. Engrg.*, Ottawa, 4(3), 345-362.

Foschi, R.O. (2000). “Modeling the hysteretic response of mechanical connections for wood structures.” *Proc., World Conf. On Timber Engrg*, Whistler, Canada.

Gupta, A.K., and Kuo, G.P. (1985). “Behavior of wood-framed shear walls.” *J. Struct. Engrg, ASCE*, 111(8), 1722-1733.

Gupta, A.K., and Kuo, G.P. (1987). “Wood-framed shear walls with uplifting.” *J. Struct. Engrg, ASCE*, 113(2), 241-259.

Gutkowski, R.M. and Castillo, A.L. (1988). “Single- and double-sheathed wood shear wall study.” *J. Struct. Engrg, ASCE*, 114(2), 1268-1284.

He, M., Lam, F., and Prion, H.G.L. (1998). “Influence of cyclic test protocols on performance of wood-based shear walls.” *Can. J. of Civ. Engrg.*, Ottawa, 25(3), 539-550.

He, M., Magnusson, H., Lam, F., and Prion, H.G.L. (1999). “Cyclic performance of perforated wood shear walls with over sized panels.” *J. Struct. Engrg, ASCE*, 125(1), 10-18.

ISO (1999). “Timber structures – joints made with mechanical fasteners – quasi-static reversed-cyclic test method.” *Draft document ISO TC 165/SC N*.

Itani, R.Y. and Cheung, C.K. (1984). “Nonlinear analysis of sheathed wood diaphragms.” *J. Struct. Engrg, ASCE*, 110(9), 2137-2147.

Karacabeyli, E. and Ceccotti, A. (1996). “Test results on the lateral resistance of nailed shear walls.” *Proc., Intl. Wood Engrg Conf.*, New Orleans, Volume 2, 179-186.

Krawinkler, Parisi, F., Ibarra, L., Ayoub, A., and Medina, R. (2000). “Development of a testing protocol for wood frame structures.” CUREe Publication No. W-02, Richmond, CA.

Lam, F., Prion, H.G.L., and He, M. (1997). “Lateral resistance of wood shear walls with large sheathing openings.” *J. Struct. Engrg, ASCE*, 123(12), 1666-1673.

Ramm, E. (1981). “Strategies for tracing the nonlinear response near limit points.” *Nonlinear Finite Element Analysis in Structural Mechanics*, W. Wunderlich et al., eds., Springer-Verlag, Berlin, 63-89.

Rose, J.D. (1994). “Performance of wood structural panel shear walls under cyclic (reversed) loading.” *Proc., Research Needs Workshop on Analysis, Design and Testing of Timber Structures Under Seismic Loads*, G.C. Foliente, ed., Forest Products Laboratory, University of California, Richmond, CA., 129-141.

Shenton, H.W., Dinehart D.W., and Elliott, T.E. (1998). “Stiffness and energy degradation of wood frame shear walls.” *Can. J. of Civ. Engrg.*, Ottawa, 25(3), 412-423.

Skaggs, T.D. and Rose, J.D. (1996). "Cyclic load testing of wood structural panel shear walls." *Proc., Intl. Wood Engrg Conf.*, New Orleans, Volume 2, 195-200.

Stewart, W.G. (1987). "The seismic design of plywood sheathed shear walls." Ph.D. thesis, University of Canterbury, Christchurch , New Zealand.

Tuomi, R.L. and McCutcheon, W.J. (1978). "Racking strength of light-frame nailed walls walls." *J. Struct. Engrg, ASCE*, 104(7), 1131-1140.

White, M.W. and Dolan, J.D. (1995). "Non-linear shear-wall analysis." *J. Struct. Engrg, ASCE*, 121(11), 1629-1635.

APPENDIX A: EVALUATION OF THE GLOBAL STIFFNESS MATRICES

The non-linear governing equilibrium equations for the cyclic response of a shear wall assembly were given by Eq. (14):

$$\boldsymbol{K_S}\,\boldsymbol{D} = \boldsymbol{F} \tag{14}$$

where $\boldsymbol{K_s} = \boldsymbol{K_s(D)}$, $\boldsymbol{D}$ and $\boldsymbol{F}$ are, respectively, the global secant stiffness matrix, displacement vector and force vector. The evaluation of $\boldsymbol{K_s}$ yields the following symmetric non-banded matrix for a wall with $\boldsymbol{N_p}$ sheathing panels and $\boldsymbol{N_c}$ connectors per sheathing panel:

$$\boldsymbol{K_S} = \left[\begin{array}{cccccccccccccc}
\sum_{i=1}^{N_p} K_{11}^{1} & K_{12}^{1} & K_{13}^{1} & K_{14}^{1} & K_{15}^{1} & K_{12}^{2} & K_{13}^{2} & K_{14}^{2} & K_{15}^{2} & \cdots & K_{12}^{N_p} & K_{13}^{N_p} & K_{14}^{N_p} & K_{15}^{N_p} \\
 & K_{22}^{1} & K_{23}^{1} & K_{34}^{1} & K_{35}^{1} & & & & & & & & & \\
 & & K_{33}^{1} & K_{34}^{1} & K_{35}^{1} & & \boldsymbol{0} & & & & & \boldsymbol{0} & & \\
 & & & K_{44}^{1} & K_{45}^{1} & & & & & & & & & \\
 & & & & K_{55}^{1} & & & & & & & & & \\
 & & & & & K_{22}^{2} & K_{23}^{2} & K_{24}^{2} & K_{25}^{2} & & & & & \\
 & & & & & & K_{33}^{2} & K_{34}^{2} & K_{35}^{2} & & & \boldsymbol{0} & & \\
 & & & & & & & K_{44}^{2} & K_{45}^{2} & & & & & \\
 & & & & & & & & K_{55}^{2} & & & & & \\
 & & & \text{Symmetric} & & & & & & \ddots & & & & \\
 & & & & & & & & & & K_{22}^{N_p} & K_{23}^{N_p} & K_{24}^{N_p} & K_{25}^{N_p} \\
 & & & & & & & & & & & K_{33}^{N_p} & K_{34}^{N_p} & K_{35}^{N_p} \\
 & & & & & & & & & & & & K_{44}^{N_p} & K_{45}^{N_p} \\
 & & & & & & & & & & & & & K_{55}^{N_p}
\end{array}\right] \tag{24}$$

with coefficients

$$\boldsymbol{K_{11}^{i}} = \frac{\boldsymbol{1}}{\boldsymbol{H^2}} \sum_{j=1}^{N_c} \boldsymbol{k_{ij}^{(s)}} \left(\boldsymbol{y_{ij}} + \boldsymbol{\bar{y}_i} \right)^2 \tag{25a}$$

$$K_{12}^{i} = \frac{-2}{Hh_i}\sum_{j=1}^{N_c} k_{ij}^{(s)} y_{ij}\left(y_{ij} + \bar{y}_i\right) \tag{25b}$$

$$K_{13}^{i} = \frac{-1}{H}\sum_{j=1}^{N_c} k_{ij}^{(s)}\left(y_{ij} + \bar{y}_i\right) \tag{25c}$$

$$K_{15}^{i} = \frac{1}{H}\sum_{j=1}^{N_c} k_{ij}^{(s)} y_{ij}\left(y_{ij} + \bar{y}_i\right) \tag{25d}$$

$$K_{22}^{i} = \frac{4}{h_i^2}\left[G_i b_i h_i t_i + \sum_{j=1}^{N_c} k_{ij}^{(s)} y_{ij}^2\right] \tag{25e}$$

$$K_{23}^{i} = \frac{2}{h_i}\sum_{j=1}^{N_c} k_{ij}^{(s)} y_{ij} \tag{25f}$$

$$K_{25}^{i} = \frac{-2}{h_i}\sum_{j=1}^{N_c} k_{ij}^{(s)} y_{ij}^2 \tag{25g}$$

$$K_{33}^{i} = -\sum_{j=1}^{N_c} k_{ij}^{(s)} \tag{25h}$$

$$K_{35}^{i} = \sum_{j=1}^{N_c} k_{ij}^{(s)} y_{ij} \tag{25i}$$

$$K_{44}^{i} = K_{33}^{i} \tag{25j}$$

$$K_{45}^{i} = \sum_{j=1}^{N_c} k_{ij}^{(s)} x_{ij} \tag{25k}$$

$$K_{55}^{i} = \sum_{j=1}^{N_c} k_{ij}^{(s)}\left(x_{ij}^2 + y_{ij}^2\right) \tag{25l}$$

$$K_{15}^{i} = K_{24}^{i} = K_{34}^{i} = 0 \tag{25m}$$

The term $k_{ij}^{(s)}$, which appears in each non-zero coefficient of Eq. (24) represents the secant stiffness of the j-th connector in the i-th sheathing panel, locally located within the panel at coordinates (x_{ij}, y_{ij}). In the stiffness matrix coefficients given by Eq. (25a) to Eq. (25m), each sheathing-to-framing connector has been represented by a single non-linear spring. As noted previously, to obtain the wall response under general cyclic loading each sheathing-to-framing connector in this model is represented by two orthogonal uncoupled non-linear springs. As shown in Fig. 6b, these springs are oriented parallel to the ***u***-axis and ***v***-axis and have respective stiffnesses k_u and k_v. . When two non-linear springs per connector are used, the corresponding coefficients in the global secant stiffness matrix become:

$$K_{11}^i = \frac{1}{H^2}\sum_{j=1}^{N_c}(k_u)_{ij}^{(s)}\left(y_{ij}+\bar{y}_i\right)^2 \tag{26a}$$

$$K_{12}^i = \frac{-2}{Hh_i}\sum_{j=1}^{N_c}(k_u)_{ij}^{(s)}\,y_{ij}\left(y_{ij}+\bar{y}_i\right) \tag{26b}$$

$$K_{13}^i = \frac{-1}{H}\sum_{j=1}^{N_c}(k_u)_{ij}^{(s)}\left(y_{ij}+\bar{y}_i\right) \tag{26c}$$

$$K_{15}^i = \frac{1}{H}\sum_{j=1}^{N_c}(k_u)_{ij}^{(s)}\,y_{ij}\left(y_{ij}+\bar{y}_i\right) \tag{26d}$$

$$K_{22}^i = \frac{4}{h_i^2}\left[G_i b_i h_i t_i + \sum_{j=1}^{N_c}(k_u)_{ij}^{(s)}\,y_{ij}^2\right] \tag{26e}$$

$$K_{23}^i = \frac{2}{h_i}\sum_{j=1}^{N_c}(k_u)_{ij}^{(s)}\,y_{ij} \tag{26f}$$

$$K_{25}^i = \frac{-2}{h_i}\sum_{j=1}^{N_c}(k_u)_{ij}^{(s)}\,y_{ij}^2 \tag{26g}$$

$$K_{33}^{i} = \sum_{j=1}^{N_c} (k_u)_{ij}^{(s)} \tag{26h}$$

$$K_{35}^{i} = -\sum_{j=1}^{N_c} (k_u)_{ij}^{(s)} y_{ij} \tag{26i}$$

$$K_{44}^{i} = K_{33}^{i} \tag{26j}$$

$$K_{45}^{i} = \sum_{j=1}^{N_c} (k_v)_{ij}^{(s)} x_{ij} \tag{26k}$$

$$K_{55}^{i} = \sum_{j=1}^{N_c} \left[(k_u)_{ij}^{(s)} x_{ij}^{2} + (k_v)_{ij}^{(s)} y_{ij}^{2} \right] \tag{26l}$$

$$K_{15}^{i} = K_{24}^{i} = K_{34}^{i} = 0 \tag{26m}$$

The solution strategy adopted in this study as given by Eq. (18), requires the evaluation of the global tangent stiffness matrix $K_T = K_T(D)$ for the shear wall. This matrix takes the same form as the secant stiffness matrix presented above with the exception that the term $k_{ij} = k_{ij}^{(T)}$ is the tangent stiffness of the j-th connector in the i-th panel.

In the evaluation of the global stiffness matrix, whether it is the secant or tangent, determination of k_{ij} is obtained from the hysteretic model of the connector. For the secant stiffness, $k_{ij}^{(s)}$ is obtained as the ratio of resulting total spring force to the prescribed total spring deformation through Eq. (6). For the tangent stiffness, $k_{ij}^{(T)}$ is obtained from the instantaneous slope along the current branch of the hysteretic model.

PART 2

CASHEW PROGRAM USER MANUAL

2-1. INTRODUCTION

The CASHEW program numerically evaluates the load-displacement response and energy dissipation characteristics of light-frame wood shear walls, with and without opening, under quasi-static cyclic loading. The theory underlying the development of this program is presented in Part 1 of this document. The CASHEW program has four analysis options:

1. Monotonic pushover analysis performed up to Δ_m, defined as the displacement at which the load in the wall falls to 80% of the wall's ultimate load-carrying capacity.
2. Monotonic analysis, as given by Option 1 above, followed by the CUREe-Caltech Woodframe Project testing protocol, scaled to $\Delta = 0.6\ \Delta_m$. See Part 1 - Section 1-4, for back ground information on this loading protocol.
3. Monotonic analysis, as given by Option 1 above, followed by the CUREe-Caltech Woodframe Project testing protocol, scaled to a user-specified Δ.
4. Monotonic analysis, as given by Option 1 above, followed by a user-specified loading protocol.

As part of analysis options 1-3, the load-displacement response predicted by CASHEW is used to identify the system parameters of the shear wall when modeled as an equivalent SDOF hysteretic shear element, as discussed in Part 1 – Section 1-2.6. For all analysis options the energy absorbed by the wall under the loading history is also computed.

Part 2 of this document outlines the specifications of the CASHEW program followed by detailed instructions for creating an input data file to run CASHEW. Also included is a sample data file.

2-2. CASHEW PROGRAM SPECIFICATIONS

The CASHEW 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 2-2.4 of this report. Execution of the CASHEW program can be initiated in a number of different ways:

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

Once CASHEW 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 ***filename.dat*** must have at most 12 alphanumeric characters and must include the ***.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 CASHEW performs the analysis according to the instructions specified in the data file. Upon completion of the analysis CASHEW writes to disk the following output files in the same location as the data file:

filename.out echoes the data input and summarizes the key results obtained from the analysis.

filename.pro records the loading protocol used for the analysis (applies only to analysis options 2-4). Data output is in two columns: the load step is given in column 1 and the corresponding prescribed lateral wall displacement is in column 2.

filename.mon records the load-displacement response of the wall resulting from the monotonic pushover analysis. Data output is in two columns: lateral displacement of the wall is in column 1 and the corresponding lateral load is in column 2.

filename.cyc records the load-displacement response of the wall resulting from the cyclic analysis. Data output is in two columns: lateral displacement of the wall is in column 1 and the corresponding lateral load is in column 2.

filename.eng records the energy absorbed by the wall from the applied load. Data output is in two columns: the load step is recorded in column 1 and the corresponding absorbed energy is given in column 2.

filename.sdf records the load-displacement response of the wall when modeled as an equivalent SDOF system for the same loading protocol (applies only to analysis options 1-3). Data output is in two columns: lateral displacement of the wall is in column 1 and the corresponding lateral load is in column 2.

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

- Maximum number of sheathing panels (MP) = 10
- Maximum number of horizontal connector lines per sheathing panel (ML) = 10
- Maximum number of vertical connector lines per sheathing panel (ML) = 10
- Maximum number of connectors per horizontal connector line (MC) = 50
- Maximum number of connectors per vertical connector line (MC) = 50
- Maximum number of displacement points defining a loading protocol (MDAT) = 20 000

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

As compiled CASHEW assigns only one set of sheathing-to-framing connector properties to each sheathing panel. This restriction can be relaxed through minor modifications to the source code.

2-3. CASHEW DATA FILE – GENERAL INPUT PROCEDURES

A data file, defining the problem to be analyzed, must be created in advance of running CASHEW. The user will be prompted by CASHEW 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.

2-3.1. Data Format

All input data required by CASHEW 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:

```
IDATA  RDATA  CDATA
```

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.

2-3.2. Consistent Units

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

2-3.3. Numbering of Components

Data associated with multiple shear wall components (*eg*. sheathing panels) must be entered sequentially, starting with component 1.

2-3.4. Overview of the Data Input for CASHEW

The analysis data for a particular shear wall configuration and loading protocol are specified by the following sequence of input lines, which are described in detail in the subsequent section:

Section	Description of Data Input
2-4.1.	Title for the analysis – one line.
2-4.2.	Analysis control parameter- one line.
2-4.3.	Shear wall configuration – one line.
2-4.4.	Sheathing panel geometry and material properties– one line for each sheathing panel.
2-4.5.	Sheathing-to-framing connector properties – three line for each panel.
2-4.6.	Sheathing-to-framing connector placement – one line for each horizontal connector line in a panel and one line for each vertical connector line in a sheathing panel.
2-4.7.	Reference displacement for user-defined loading protocol – one line.
2-4.8.	Number of displacement steps in the loading protocol – one line.
2-4.9.	User defined displacement loading protocol – one line for each load step

2-4. CASHEW DATA FILE - INSTRUCTIONS

2-4.1. Title for the Analysis

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

2-4.2. Analysis Control Parameter

IANALY

IANALY	=	0	Data is checked and echoed to file ***filename.out***, after which program execution is terminated.
	=	1	Monotonic analysis **only** is performed up to Δ_m, defined as the displacement at which the load in the wall falls to 80% of the wall's ultimate load-carrying capacity.
	=	2	Monotonic analysis followed by the CUREe-Caltech Woodframe Project loading protocol, scaled to $\Delta = 0.6\ \Delta_m$.
	=	3	Monotonic analysis followed by the CUREe-Caltech Woodframe Project loading protocol, scaled to a user-specified Δ.
	=	4	Monotonic analysis followed by a user-specified loading protocol.

2-4.3. Shear Wall Configuration

HTWALL NPANEL

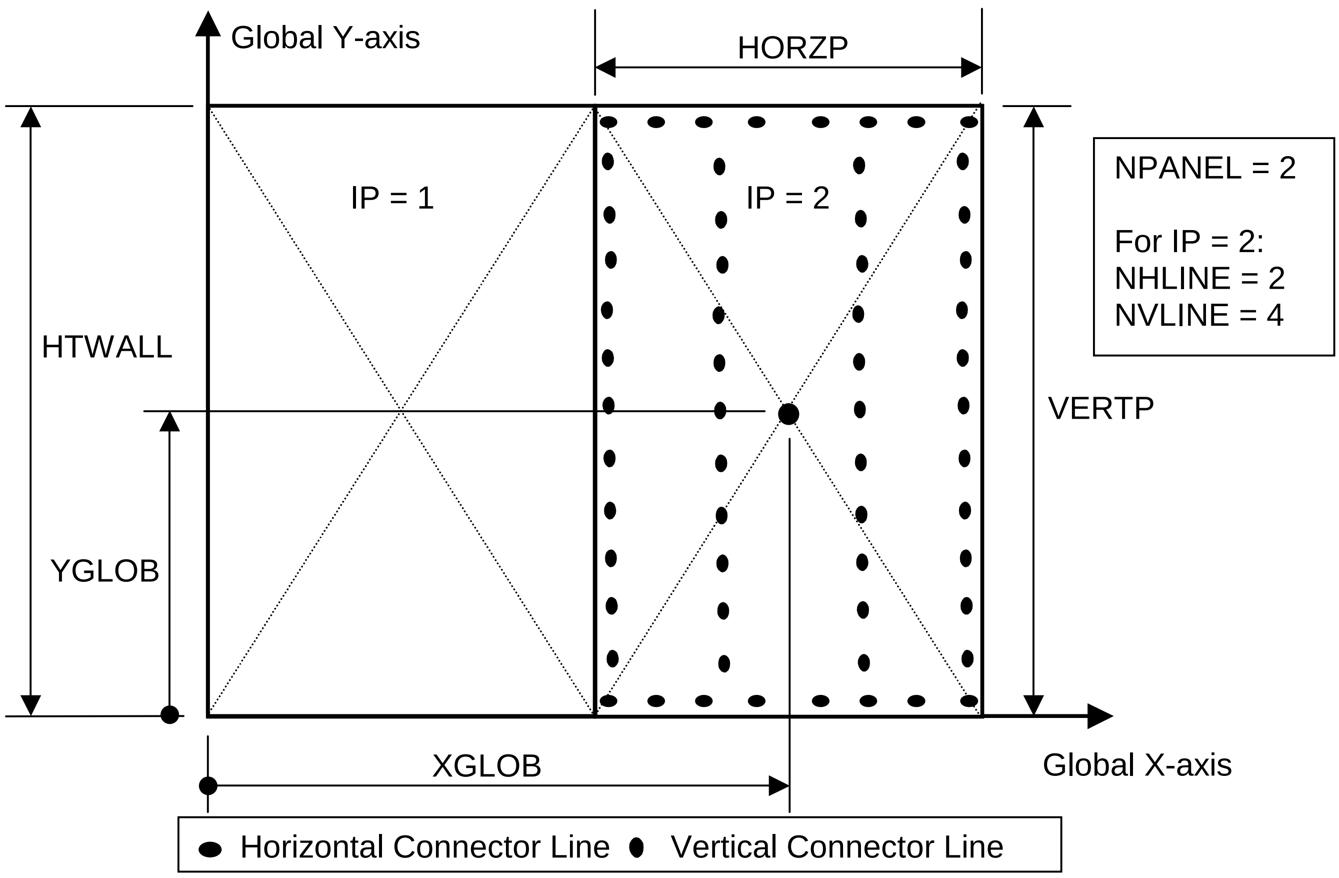
HTWALL	Height of the shear wall, measured from the sill plate to the centerline of the top plate, which is the point of application of the lateral load.
NPANEL	Number of rectangular sheathing panels in the shear wall. (NPANEL $\leq$ 10).

2-4.4. Sheathing Panel Geometry and Material Properties

One input line is required for each sheathing panel. NPANEL lines in total.

```
IP  HORZP  VERTP  THICKP  XGLOG  YGLOB  NHLINE  NVLINE  GMOD
```

IP	Sheathing panel number.
HORZP	Horizontal length of the sheathing panel.
VERTP	Vertical height of the sheathing panel.
THICKP	Thickness of the sheathing panel.
XGLOB	Global x-coordinate to locate the centroid of the sheathing panel.
YGLOB	Global y-coordinate to locate the centroid of the sheathing panel.
NHLINE	Number of horizontal sheathing-to-framing connector lines in the sheathing panel. (NHLINE ≤ 10).
NVLINE	Number of vertical sheathing-to-framing connector lines in the sheathing panel. (NVLINE ≤ 10).
GMOD	Elastic shear modulus of the sheathing panel.



Typical Shear Wall Configuration.

2-4.5. Sheathing-to-Framing Connector Properties

The four input steps in Section 2-4.5, as a block, must be repeated NPANEL times.

```
IP
```

IP Sheathing panel number.

One input line is required for each sheathing panel.

```
F0  FI  DU
```

F0 Intercept connector strength for the asymptotic line to the envelope curve. (F0 > FI > 0).
FI Intercept connector strength for the pinching branch of the hysteretic curve. (FI > 0).
DU Connector displacement at ultimate load. (DU > 0).

One input line is required for each sheathing panel.

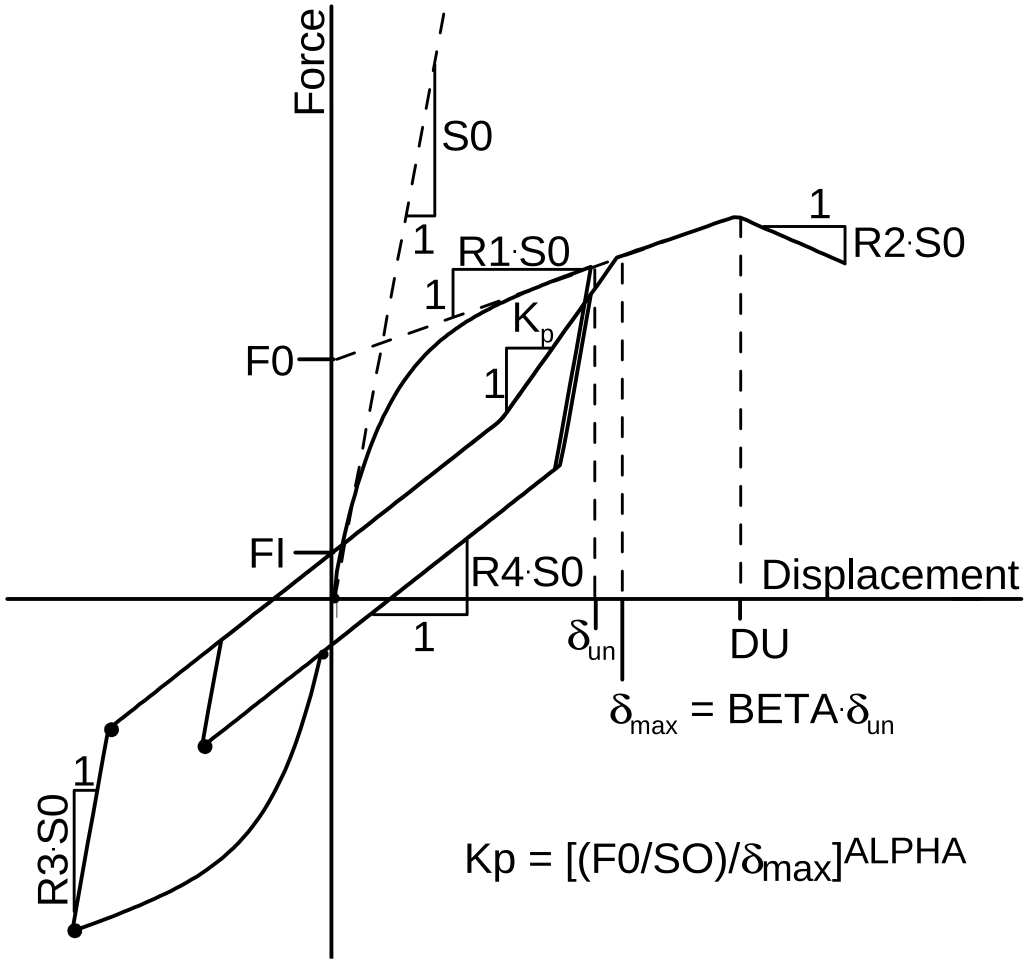
```
S0  R1  R2  R3  R4
```

S0 Initial connector stiffness (S0 > 0).
R1 Stiffness ratio of the asymptotic line to the connector envelope curve. The slope of this line is R1·S0. (0 < R1 < 1.0).
R2 Stiffness ratio of the descending branch of the connector envelope curve. The slope of this line is R2·S0. (R2 < 0).
R3 Stiffness ratio of the unloading branch off the connector envelope curve. The slope of this line is R3·S0. (R3 > 0).
R4 Stiffness ratio of the pinching branch for the connector. The slope of this line is R4·S0. (R4 > 0).

One input line is required for each sheathing panel.

```
ALPHA  BETA
```

ALPHA Stiffness degradation connector parameter. (ALPHA > 0).
BETA Stiffness degradation connector parameter. (BETA > 0).



Hysteretic Response of a Sheathing-to-Framing Connector.

2-4.6. Sheathing-to-Framing Connector Placement

The three input steps in Section 2-4.6, as a block, must be repeated NPANEL times.

```
IP
```

IP — Sheathing panel number.

One input line is required for each horizontal connector line in the sheathing panel. Input continues in turn for each horizontal connector line up to NHLINE.

```
YLOCAL  XSTART  XEND  SPACEH
```

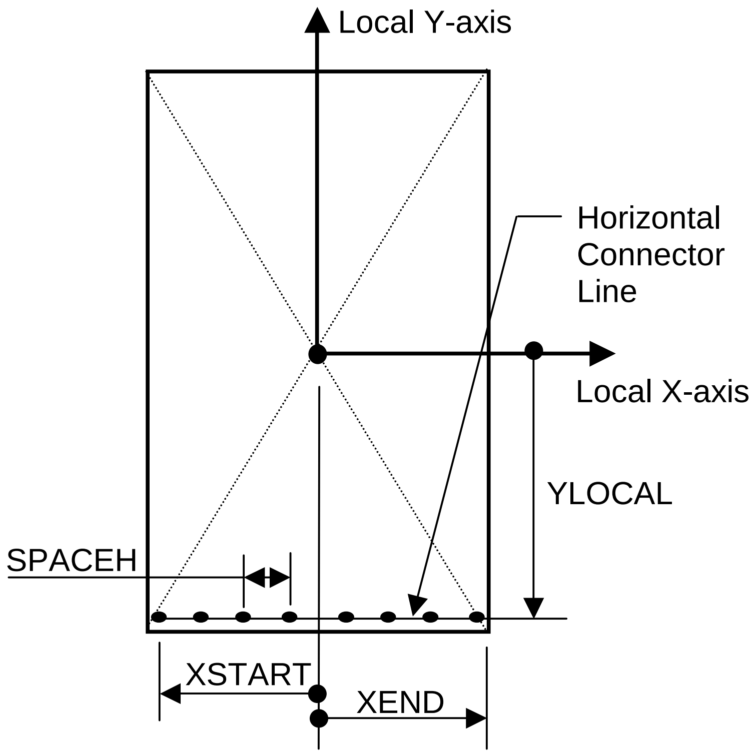
YLOCAL	Local y-coordinate of the horizontal connector line. YLOCAL is measured with respect to the centroid of the sheathing panel.
XSTART	Local x-coordinate indicating the start of the horizontal connector line. XSTART is measured with respect to the centroid of the sheathing panel.
XEND	Local x-coordinate indicating the end of the horizontal connector line. XEND is measured with respect to the centroid of the sheathing panel. (XEND > XSTART)
SPACEH	Spacing between sheathing-to-framing connectors along the horizontal connector line.

One input line is required for each vertical connector line in the sheathing panel. Input continues in turn for each vertical connector line up to NVLINE.

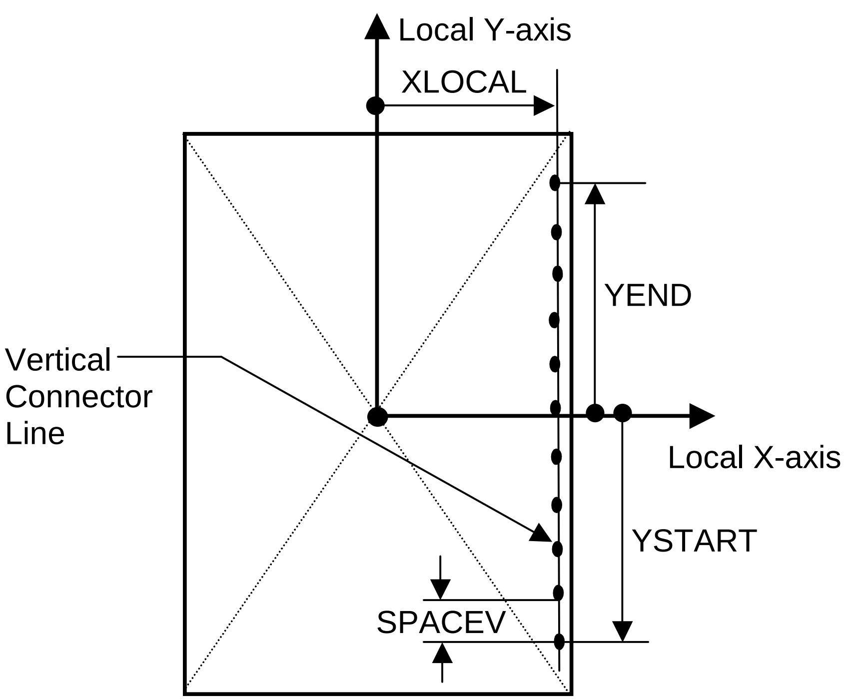
```
XLOCAL  YSTART  YEND  SPACEV
```

XLOCAL	Local x-coordinate of the vertical connector line. XLOCAL is measured with respect to the centroid of the sheathing panel.
YSTART	Local y-coordinate indicating the start of the vertical connector line. YSTART is measured with respect to the centroid of the sheathing panel.
YEND	Local y-coordinate indicating the end of the vertical connector line. YEND is measured with respect to the centroid of the sheathing panel. (YEND > YSTART).
SPACEV	Spacing between sheathing-to-framing connectors along the vertical connector line.

Note: If a single connector occurs at the intersection of a horizontal and vertical connector line it must be associated with only one of the connector lines. This point is illustrated in the following Figure, where the corner connector has been included in the horizontal connector line and excluded from the vertical connector line.



Sheathing-to-Framing Connector Placement for Horizontal Connector Lines.



Sheathing-to-Framing Connector Placement for Vertical Connector Lines.

2-4.7. User-Defined Reference Displacement for the CUREe-Caltech Testing Protocol

If IANALY = 3 then enter this data line.

GDELTA

GDELTA — Reference displacement to scale the CUREe-Caltech Woodframe Project testing protocol. See Fig. 12 on Page 27 and set GDELTA = Δ. (GDELTA > 0).

2-4.8. User-Defined Displacement Loading Protocol

If IANALY = 4 then enter this data line.

NDISP

NDISP — Number of data points in user-defined loading protocol. (NDISP ≤ 20,000).

If IANALY = 4 then enter this data line. One input line is required for each loading protocol data point. Input continues NDISP times to enter the loading protocol data.

GD1

GD1 — Displacement data point from the user-defined loading protocol.

2-5. CASHEW EXECUTION ERRORS AND PROGRAM TERMINATION

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

The CASHEW program may also prematurely terminate an analysis before the end of a loading protocol for one of the following three reasons:

1. Not being able to converge to an equilibrium state within a specified number of iterations;
2. Having the global stiffness matrix become singular;
3. Encountering too large of a displacement step for a connector 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 CASHEW will terminate the analysis. Possible reasons for premature program termination may be linked to incorrect data input or to not specifying a sufficiently refined loading protocol (particularly under analysis option IANALY = 3). It may also happen that the analysis terminates as a result of the load-carrying capacity of the wall been exhausted by the applied loading protocol.

2-6. EXAMPLE CASHEW DATA FILE AND SUMMARY OUTPUT FILE

The following data file was created for the shear wall described in Part 1 - Section 3. With this data file the shear wall is subjected to the CUREe-Caltech loading protocol (IANALY = 2).

<u>EXAMPLE.DAT</u>

```
2.4m X 2.4m OSB Sheathed UBC Test Shear Wall, Units are kN - mm
2,                                        ! Analysis Control Parameter
2440.,3,                                  ! Height of wall, Number of panels
1,2400.,1180.,9.5,1220.,610.0,2,7,1.5,    ! Panel 1 geometric & material props.
2,1180.,1180.,9.5,590.0,1830.,2,4,1.5,    ! Panel 2 geometric & material props.
3,1180.,1180.,9.5,1810.,1830.,2,4,1.5,    ! Panel 3 geometric & material props.
1,                                        ! Panel 1 connector properties
0.751,0.141,12.5,                         !
0.561,0.061,-0.078,1.40,0.05,             !
0.80,1.1,                                 !
2                                         ! Panel 2 connector properties
0.751,0.141,12.5,                         !
0.561,0.061,-0.078,1.40,0.05,             !
0.80,1.1,                                 !
3                                         ! Panel 3 connector properties
0.751,0.141,12.5,                         !
0.561,0.061,-0.078,1.40,0.05,             !
0.80,1.1,                                 !
1                                         ! Panel 1 connector placement
-590.0,-1180.0,1180.0,147.5,              ! Horizontal connector lines
590.00,-1180.0,1180.0,147.5,              !
-1200.,-446.25,446.25,147.5,              ! Vertical connector lines
-800.0,-295.00,295.00,295.0,              !
-400.0,-295.00,295.00,295.0,              !
0.0000,-295.00,295.00,295.0,              !
400.00,-295.00,295.00,295.0,              !
800.00,-295.00,295.00,295.0,              !
1200.0,-446.25,446.25,147.5,              !
2                                         ! Panel 2 connector placement
-590.,-590.00,590.00,147.5,               ! Horizontal connector lines
590.0,-590.00,590.00,147.5,               !
-590.,-446.25,446.25,147.5,               ! Vertical connector lines
-190.,-295.00,295.00,295.0,               !
210.0,-295.00,295.00,295.0,               !
590.0,-446.25,446.25,147.5,               !
3                                         ! Panel 3 connector placement
-590.,-590.00,590.00,147.5,               ! Horizontal connector lines
590.0,-590.00,590.00,147.5,               !
-590.,-446.25,446.25,147.5,               ! Vertical connector lines
-190.,-295.00,295.00,295.0,               !
210.0,-295.00,295.00,295.0,               !
590.0,-446.25,446.25,147.5,               !
```

The associated summary output file produced by CASHEW is given below:

<u>EXAMPLE.OUT</u>

```
          *-------------------------------------------------------*
          |                                                       |
          |                        CASHEW                         |
          |                                                       |
          |             Cyclic Analysis of SHEarWalls             |
          |                     Version 1.0                       |
          |                                                       |
          |           Structural Engineering Department           |
          |          University of California, San Diego          |
          |                     October 2000                      |
          |                                                       |
          *-------------------------------------------------------*


  Problem: 2.4m X 2.4m OSB Sheathed UBC Test Shear Wall, Units are kN - mm

  Solution option: Cyclic analysis under the CUREe loading protocol.

  Total wall height =    0.244000E+04
  Number of sheathing panelS =  3

  Sheathing panel geometry:
  Panel       Width       Height       Thickness       Global        Global
   No.                                              x-coordinate   y-
coordinate
    1     0.240000E+04  0.118000E+04  0.950000E+01  0.122000E+04
0.610000E+03
    2     0.118000E+04  0.118000E+04  0.950000E+01  0.590000E+03
0.183000E+04
    3     0.118000E+04  0.118000E+04  0.950000E+01  0.181000E+04
0.183000E+04

  Sheathing panel material properties:
  Panel       Shear
   No.       Modulus
    1     0.150000E+01
    2     0.150000E+01
    3     0.150000E+01

  Sheathing-to-framing connector properties:
  Panel No. =    1
  Stiffness Parameters:
       S0            R1            R2            R3            R4
  0.56100E+00   0.61000E-01  -0.78000E-01   0.14000E+01   0.50000E-01
  Strength, deformation and degradation parameters:
       F0            FI            DU         ALPHA          BETA
  0.75100E+00   0.14100E+00   0.12500E+02   0.80000E+00   0.11000E+01
  Panel No. =    2
  Stiffness Parameters:
       S0            R1            R2            R3            R4
  0.56100E+00   0.61000E-01  -0.78000E-01   0.14000E+01   0.50000E-01
  Strength, deformation and degradation parameters:
       F0            FI            DU         ALPHA          BETA
  0.75100E+00   0.14100E+00   0.12500E+02   0.80000E+00   0.11000E+01
```

```
Panel No. =    3
Stiffness Parameters:
     S0             R1             R2             R3             R4
0.56100E+00   0.61000E-01  -0.78000E-01   0.14000E+01   0.50000E-01
Strength, deformation and degradation parameters:
     F0             FI             DU          ALPHA          BETA
0.75100E+00   0.14100E+00   0.12500E+02   0.80000E+00   0.11000E+01

Sheathing-to-framing connector placement:
Panel No. =    1
Horizontal connector lines:
Line        Local     x-coordinate  x-coordinate   Connector
 No.    y-coordinate    at start      at end         spacing
  1    -0.590000E+03 -0.118000E+04  0.118000E+04  0.147500E+03
  2     0.590000E+03 -0.118000E+04  0.118000E+04  0.147500E+03
Vertical connector lines:
Line        Local     y-coordinate  y-coordinate   Connector
 No.    x-coordinate    at start      at end         spacing
  1    -0.120000E+04 -0.446250E+03  0.446250E+03  0.147500E+03
  2    -0.800000E+03 -0.295000E+03  0.295000E+03  0.295000E+03
  3    -0.400000E+03 -0.295000E+03  0.295000E+03  0.295000E+03
  4     0.000000E+00 -0.295000E+03  0.295000E+03  0.295000E+03
  5     0.400000E+03 -0.295000E+03  0.295000E+03  0.295000E+03
  6     0.800000E+03 -0.295000E+03  0.295000E+03  0.295000E+03
  7     0.120000E+04 -0.446250E+03  0.446250E+03  0.147500E+03

Sheathing-to-framing connector placement:
Panel No. =    2
Horizontal connector lines:
Line        Local     x-coordinate  x-coordinate   Connector
 No.    y-coordinate    at start      at end         spacing
  1    -0.590000E+03 -0.590000E+03  0.590000E+03  0.147500E+03
  2     0.590000E+03 -0.590000E+03  0.590000E+03  0.147500E+03
Vertical connector lines:
Line        Local     y-coordinate  y-coordinate   Connector
 No.    x-coordinate    at start      at end         spacing
  1    -0.590000E+03 -0.446250E+03  0.446250E+03  0.147500E+03
  2    -0.190000E+03 -0.295000E+03  0.295000E+03  0.295000E+03
  3     0.210000E+03 -0.295000E+03  0.295000E+03  0.295000E+03
  4     0.590000E+03 -0.446250E+03  0.446250E+03  0.147500E+03

Sheathing-to-framing connector placement:
Panel No. =    3
Horizontal connector lines:
Line        Local     x-coordinate  x-coordinate   Connector
 No.    y-coordinate    at start      at end         spacing
  1    -0.590000E+03 -0.590000E+03  0.590000E+03  0.147500E+03
  2     0.590000E+03 -0.590000E+03  0.590000E+03  0.147500E+03
Vertical connector lines:
Line        Local     y-coordinate  y-coordinate   Connector
 No.    x-coordinate    at start      at end         spacing
  1    -0.590000E+03 -0.446250E+03  0.446250E+03  0.147500E+03
  2    -0.190000E+03 -0.295000E+03  0.295000E+03  0.295000E+03
  3     0.210000E+03 -0.295000E+03  0.295000E+03  0.295000E+03
  4     0.590000E+03 -0.446250E+03  0.446250E+03  0.147500E+03

----------------------------------------------------------------------------

<> Loading protocol is recorded in the file:
   EXAMPLE.PRO
```

```
<> Monotonic load-deformation response of the wall is recorded in the file:
   EXAMPLE.MON


<> Cyclic load-deformation response of the wall is recorded in the file:
   EXAMPLE.CYC

<> Hysteretic energy absorbed during loading is recorded in the file:
   EXAMPLE.ENG

<> SDOF model parameters for the wall are recorded in the file:
   EXAMPLE.SDF

----------------------------------------------------------------------------

Analysis Summary:

Initial wall stiffness              =     0.152376E+01

Ultimate lateral load               =     0.219960E+02

Displacement @ ultimate load        =     0.600240E+02

CUREe protocol displacement DELTA   =     0.589992E+02

SDOF system ID under cyclic loading:
WS0       =   0.1441E+01
WR1       =   0.8099E-01
WR2       =  -0.2182E-01
WR3       =   0.1312E+01
WR4       =   0.6400E-01
WF0       =   0.1509E+02
WFI       =   0.3134E+01
WDULT     =   0.6002E+02
WALPHA    =   0.7426E+00
WBETA     =   0.1096E+01
```