Analytical Model for Sheathing-to-Framing Connections in Wood Shear Walls and Diaphragms

Johnn P. Judd and Fernando S. Fonseca, P.E., A.M.ASCE

Abstract: A new analytical model for sheathing-to-framing connections in wood shear walls and diaphragms is discussed in this paper. The model represents sheathing-to-framing connections using an oriented pair of nonlinear springs. Unlike previous models, the new analytical model is suitable for both monotonic and cyclic analyses and does not need to be scaled or adjusted. Furthermore, the analytical model may be implemented in a general purpose finite element program, such as ABAQUS, or in a specialized structural analysis program, such as CASHEW. To illustrate, the responses of a 4.88 × 14.6 m plywood diaphragm and a 2.44 × 2.44 m oriented strand board shear wall are predicted using the new analytical model.

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Introduction

In wood housing, lateral forces caused by earthquakes or strong winds are usually resisted by a system of wood shear walls and diaphragms (roof and/or floors). Lateral force is transferred from the roof and floors through diaphragm action to supporting shear walls and eventually into the foundation. In Fig. 1, the primary structural components of wood shear walls and diaphragms are shown. Wood framing and sheathing panels, such as plywood or oriented strand board (OSB) are connected using fasteners (nails or staples). Additionally, shear walls may employ anchorage devices and large diaphragms may require chord splice connections.

Wood shear walls and diaphragms have generally performed well during earthquakes, in terms of preserving life. In spite of this performance, the costs of building damage to wood structures—for example, in the Northridge 1994 earthquake and 1992 Hurricane Andrew—have prompted an interest in shifting design emphasis from life safety to damage control (Rosowsky and Ellingwood 2002). Although the design philosophy of the current codes in North America has not changed from life safety, limiting structural damage may become a primary objective of next-generation performance-based design procedures (FEMA 2000). For wood structures, performance-based design may more precisely be termed displacement-based design because the primary objective is to limit interstory drift.

Displacement-based design is considered to have a number of advantages compared to conventional force-based design (Filiatrault and Folz 2002). In the conventional forced-based design, the force required so that wood structures remain elastic is determined. The design force is then obtained by dividing the elastic force by a reduction factor R, which is used to account for structural ductility. The R factor is difficult to determine, however, because wood structures behave inelasticity, even at lower load levels. In a displacement-based design, the structure must meet a target displacement (such as interstory drift) instead of a force requirement. Thus, neither an elastic estimate of the structure, nor a reduction factor is necessary.

A displacement-based design requires an understanding of the pushover (monotonic) response of the structure. This knowledge can be acquired through experimental testing and structural analysis. Although experimental testing cannot be completely replaced, executing a structural analysis computer program is typically less expensive and less time consuming compared to testing.

For wood shear walls, a variety of structural analysis tools are available. The most simple tools consist of a single-degree-of-freedom (SDOF) system (Medearis 1970; Stewart 1987; Foliente 1995; van de Lindt and Waltz 2003). In a SDOF system, the relationship between the applied force and lateral displacement at the top of a shear wall is calibrated to data from experimental testing. An advantage of using a SDOF system is that it may easily be employed in a subsequent dynamic analysis. Nevertheless, SDOF systems are limited to the specific materials and configurations used to calibrate the model, and are seldom used for wood diaphragm analysis.

A number of specialized structural analysis programs for wood shear walls have been developed based on the understanding that the overall lateral behavior is dominated by the individual behavior of sheathing-to-framing connections (Tuomi and McCutcheon 1978; Gupta and Kuo 1985, 1987; Filiatrault 1990; Dinehart and Shenton 2000; Folz and Filiatrault 2000; Richard et al. 2002). In these structural analysis programs, sheathing-to-framing connections are represented using a single nonlinear spring or a pair of orthogonal nonlinear springs. In general, wood framing is assumed to be rigid and pin connected, and all sheathing panels are assumed to undergo the same rotation and translation. It is important to note that this latter assumption is not valid for wood dia-
phragms, however, since sheathing panels near the midspan may rotate less relative to panels near the supports.

The finite element method has been successfully used to develop structural analysis programs for both wood shear walls and diaphragms (Easley et al. 1982; Itani and Cheung 1984; Dolan and Foschi 1991; White and Dolan 1995; Fonseca 1997; He et al. 2001; Symans et al. 2001; Hite and Shenton 2002). Wood framing is represented using standard linear beam elements. Sheathing panels, insulation, and exterior (stucco) and interior (gypsum wall board) finish materials, if included, may be represented using linear plane-stress elements. Sheathing-to-framing connections are represented using nonlinear spring elements, and chord splices are represented using linear spring elements. An advantage of a finite element analysis is an increased understanding of force distribution between structural components. A disadvantage is that the amount of information and detailed computer modeling required are cumbersome for routine design. Besides, for wood shear walls, the more sophisticated finite element analysis programs yield approximately the same accuracy as the simpler specialized structural analysis programs (Folz and Filiatrault 2001).

An overarching concern with currently available structural analysis programs is the lack of a rigorous analytical model for sheathing-to-framing connections. Structural analysis programs that represent sheathing-to-framing connections using one nonlinear spring (single spring model) are incapable of reversed cyclic loading (Folz and Filiatrault 2001). This limitation is significant because reversed cyclic loading is required to determine energy dissipation characteristics. Furthermore, structural analysis programs using a single spring model may be unstable, especially near ultimate loading.

Structural analysis programs that represent sheathing-to-framing connections using two orthogonal nonlinear springs (non-oriented spring pair model) overestimate connection strength and stiffness. For wood shear walls, Folz and Filiatrault (2001) proposed a method to compensate for the overestimation of strength and stiffness using the structural analysis program CASHEW (Folz and Filiatrault 2000). In CASHEW, the sheathing-to-framing connection spacing is scaled, or adjusted, internally within the computer program until the energy absorbed by the wall using a nonoriented spring pair model agrees with the energy absorbed by the wall using a single spring model. Although this method successfully compensates for the overestimation for wood shear walls, it is not a feasible method for many structural analysis programs, such as a general purpose finite element program. The objective of this paper is to provide a rigorous analytical model for sheathing-to-framing connections that does not need to be scaled or adjusted.

**Actual Behavior of Sheathing-to-Framing Connections**

Lateral deformation of a basic panel section in a wood shear wall or diaphragm is depicted in Fig. 2. In the undeformed configuration, the location of a specific fastener (nail) head in the sheathing panel, point A, is coincident with the location of the same nail shank embedded in the wood framing, point B. Thus, in Fig. 2, points A and B are the same points. During lateral loading, the specific nail head displaces from point A to point A’. The nail shank embedded in the wood framing displaces from point B to point B’. Because the displacement of the sheathing is not necessarily equal to the displacement of the framing, due to the shear strength of the sheathing, point A’ and point B’ are not coincident.

In Fig. 3, the lateral deformation of a specific sheathing-to-framing connection is depicted. The lateral force (connection force \( P \)) transferred from the sheathing through the nail displaces the nail head relative to the nail shank (connection displacement \( \Delta \)). Initially, as the nail head displaces and the nail shank deforms, the force–displacement relationship is linear. The wood fibers, sheathing, and nail all remain elastic. As loading progresses, the displacement of the connection increases, the wood fibers crush, and the nail may yield. The angle of the applied lateral load with respect to the wood grain has a negligible effect on the connection behavior (Dolan and Madsen 1992). If the loading is reversed, the nail moves through the gap formed by the crushed wood fibers and the connection exhibits low stiffness and strength until the nail again comes into contact with the wood. In Fig. 4, a typical force–displacement relationship for a sheathing-to-framing connection subjected to reversed-cyclic loading is shown. The primary characteristics of the relationship are pinched hysteresis loops (pinching behavior), inelastic behavior, and strength and...

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**Fig. 1.** Primary structural components of wood shear walls and diaphragms

**Fig. 2.** Lateral deformation of a basic panel section in a wood shear wall or diaphragm

**Fig. 3.** Lateral deformation of a sheathing-to-framing connection
stiffness degradation. If the loading continues after yielding of the nail, prior to failure the strength of the connection decreases with increasing displacement.

**Idealized Behavior of Sheathing-to-Framing Connections**

The force–displacement relationship for unidirectional loading may be idealized by modeling the connection as an elastoplastic pile (fastener) embedded into a nonlinear layered medium (wood framing and sheathing). In this approach, the mechanical properties of the sheathing, framing, and fastener are required (Foschi 2000).

Alternatively, the force–displacement relationship may be determined by experimental testing of individual sheathing-to-framing connection assemblies (coupon testing). In this approach, fastener withdrawal is considered implicitly. The force–displacement relationship during monotonic loading is idealized using a mathematical expression. During reversed-cyclic loading, the monotonic force–displacement relationship provides a response envelope, while hysteresis behavior is idealized using a predefined set of load paths to describe unloading, load reversal, and reloading.

A sheathing-to-framing connection is commonly represented in a structural analysis program as a two-node element (Fig. 5). The first node (point A') is the location of the nail head in the sheathing panel and the second node (point B') is the location of the nail shank in the wood framing. Each node has two degrees of freedom corresponding to in-plane translations. The resultant connection displacement $\Delta_r$ is calculated using the $x$-direction component $\Delta_x$ and the $y$-direction component $\Delta_y$.

**Single Spring Model**

In the single spring model, a sheathing-to-framing connection is represented using one nonlinear spring [Fig. 6(a)]. Since the displacement trajectory of a sheathing-to-framing connection is primarily unidirectional during monotonic loading (Tuomi and McCutcheon 1978), the total displacement of the connection may be estimated as the resultant displacement $\Delta_r$. Therefore, the element stiffness matrix $K$ is formulated as a “shear element,” where the spring stiffness is equal in the $x$ and $y$ directions and the nodal force vector $F$ is assumed to be proportional to the nodal displacements. The connection stiffness $K$ and the connection force $P_r$ are a function of the resultant displacement $\Delta_r$ [Fig. 6(b)]

\[
K = \begin{bmatrix}
K_x & 0 & -K_x & 0 \\
0 & K_y & 0 & -K_y \\
-K_x & 0 & K_x & 0 \\
0 & -K_y & 0 & K_y
\end{bmatrix}
\]

\[
F = \begin{bmatrix}
P_r(\Delta_x/\Delta_r) \\
P_r(\Delta_y/\Delta_r) \\
-P_r(\Delta_x/\Delta_r) \\
-P_r(\Delta_y/\Delta_r)
\end{bmatrix}
\]  

Two concerns arise when using the single spring model in a structural analysis program. The first concern is that the displacement trajectory can be bidirectional under reversed-cyclic loading or under highly nonlinear loading. As a consequence, the total connection displacement is path dependent and not necessarily equivalent to the resultant displacement $\Delta_r$. In addition, it is not
The primary concern with the nonoriented spring pair model is that it overestimates connection stiffness and force under nonlinear loading. With one spring in the $x$ direction and the other spring in the $y$ direction, the stiffness and force are arbitrary: The values of spring stiffness and force change relative to the displacement trajectory. For example, using the nonoriented spring pair model, sheathing-to-framing connections displaced along a trajectory of $45^\circ$ with respect to the $x$-direction, such as connections located near sheathing panel corners, have greater stiffness than identical connections equally displaced along a trajectory in the $x$ direction. Clearly, for a given displacement, this is incorrect. Actual connection stiffness is the same regardless of the displacement trajectory. Yet, this overestimation is not confined to connections near panel corners because deformations of connections along panel edges vary approximately in proportion to the distance from the panel corner (Tuomi and McCutcheon 1978; McCutcheon 1985; Schmidt and Moody 1989; Fonseca 1997). As a result, structural analysis programs using the nonoriented spring pair model overestimate shear wall or diaphragm strength. The magnitude of overestimation is not accurately determined a priori, however, since the overestimation is a function of the wall (or diaphragm) aspect ratio, nail spacing, nail pattern, and shear modulus of the framing and sheathing.

To compensate for this overestimation Folz and Filiatrault (2000) proposed a novel method for analysis of wood shear walls. In their method, the sheathing-to-framing connection spacing is adjusted until the energy absorbed by the wall using the nonoriented spring pair model agrees with the energy absorbed by the wall using a single spring model. Their model has subsequently been implemented into a structural analysis program (SAWS) for buildings composed of rigid horizontal diaphragms and wood shear walls (Folz and Filiatrault 2002). Although this adjustment is an ingenious solution for wood shear wall analysis, a rigorous solution is required for wood diaphragm analysis.

**Oriented Spring Pair Model**

In the oriented spring pair model, a sheathing-to-framing connection is represented using two orthogonal nonlinear springs that are oriented using the initial displacement trajectory [Fig. 8(a)]. The initial displacement trajectory in the $u$ direction may be defined using the displacement at time zero, during a time–history analysis, or the linear displacement, during a linear analysis [Fig. 8(b)]. The component of connection displacement along the initial displacement trajectory is $\Delta_u$, and the off-directional ($v$ direction) component is $\Delta_v$. The angle between the $u$ and $x$ directions is $\theta$. In this way, the element stiffness matrix $K$ and the nodal force vector $F$ are coupled in the $x$ and $y$ directions. The connection stiffnesses, $K_x$ and $K_y$, and connection forces, $P_x$ and $P_y$, are a function of the respective $u$ and $v$ direction displacements $[\Delta u, \Delta v]$. In the model, the stiffness represents the slope of the load path at a specific point

$$
K = \begin{bmatrix}
K_x & 0 & -K_x & 0 \\
0 & K_y & 0 & -K_y \\
-K_x & 0 & K_x & 0 \\
0 & -K_y & 0 & K_y
\end{bmatrix}
$$

$$
F = \begin{bmatrix}
P_x \\
P_y \\
-P_x \\
-P_y
\end{bmatrix}
$$

The primary concern with the nonoriented spring pair model is that it overestimates connection stiffness and force under nonlinear loading. With one spring in the $x$ direction and the other spring in the $y$ direction, the stiffness and force are arbitrary: The values of spring stiffness and force change relative to the displacement trajectory. For example, using the nonoriented spring pair model, sheathing-to-framing connections displaced along a trajectory of $45^\circ$ with respect to the $x$-direction, such as connections located near sheathing panel corners, have greater stiffness than identical connections equally displaced along a trajectory in the $x$ direction. Clearly, for a given displacement, this is incorrect. Actual connection stiffness is the same regardless of the displacement trajectory. Yet, this overestimation is not confined to connections near panel corners because deformations of connections along panel edges vary approximately in proportion to the distance from the panel corner (Tuomi and McCutcheon 1978; McCutcheon 1985; Schmidt and Moody 1989; Fonseca 1997). As a result, structural analysis programs using the nonoriented spring pair model overestimate shear wall or diaphragm strength. The magnitude of overestimation is not accurately determined a priori, however, since the overestimation is a function of the wall (or diaphragm) aspect ratio, nail spacing, nail pattern, and shear modulus of the framing and sheathing.

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$$
K = \begin{bmatrix}
K_{11} & K_{12} & -K_{11} & -K_{12} \\
K_{12} & K_{22} & -K_{22} & K_{12} \\
-K_{11} & -K_{22} & K_{11} & K_{22} \\
K_{sym} & & &
\end{bmatrix}
$$

where
Nails pulling, or tearing, through sheathing panels is a dominate failure mode observed during reversed-cyclic loading of wood shear walls and diaphragms (Durham 1998; Olpin 1998; Jones and Fonseca 2002). Tearing of the sheathing prevents the connection displacement trajectory from subsequently following any established path, such as a circular path, and restricts the movement of the nail to a relatively narrow path. Consequently, even though the displacement trajectory can be bidirectional, the initial displacement trajectory is primarily unidirectional.

Accordingly, in the oriented orthogonal spring pair model, the orientation is representative of actual connection behavior: The off-directional, or orthogonal spring (v direction), contribution to the element displacement is small since only the directional (u direction) spring is principally deformed. Thus, by using the oriented spring pair model, the overestimation inherent in the non-oriented spring pair model is eliminated.

The oriented spring pair model may be refined to include off-directional stiffness degradation. The reduction in off-directional stiffness, or “true” off-directional stiffness, may be determined through coupon testing. In lieu of empirical data, off-directional stiffness may be incorporated based on the deformation of the directional spring using a continuous damage function, or a discrete set of damage levels. Also, further research would be required to demonstrate implementation of the analytical model in a dynamic time-history analysis.

**Numerical Examples**

The oriented spring pair model is used in two numerical examples. In the first example, the model is implemented into a general purpose finite element program, ABAQUS (ABAQUS 2002), to predict the response of a 4.88 × 14.6 m plywood diaphragm. In the second example, the model is implemented into a specialized structural analysis program, CASHEW, to predict the response of two 2.44 × 2.44 m oriented strand board shear walls. In addition to these examples, the oriented spring pair model has also been successfully used in the analysis of shear walls with overdriven nails (Fonseca and Judd 2004) and in the analysis of roof diaphragms for seismic retrofitting (Judd and Fonseca 2003).

**Wood Diaphragm Analysis Using ABAQUS**

The response of a 4.88 × 14.6 m plywood diaphragm tested by the American Plywood Association (APA) (Tissell and Elliott 1980) is predicted using ABAQUS. These experimental results have previously been used to validate numerical models (Falk and Itani 1989; Fonseca 1997).

In the ABAQUS program, structural components are represented in a standard way (ABAQUS element type designations are shown in parentheses). Wood framing is represented using linear beam elements (B21), sheathing panels are represented using plane stress elements (CPS8R), chord splice connections are represented using linear spring elements (SPRING2), and sheathing-to-framing connections are represented using user-defined elements (U1).

The plywood diaphragm uses 12.7-mm-thick Structural I C-D plywood sheathing panels. The sheathing is attached to the framing using 3.76-mm-diameter × 76.2-mm-long nails spaced 102 mm on center along exterior panel edges, 152 mm on center along interior panel edges, and 305 mm on center in the panel fields. The framing consists of two 88.9 × 241 mm members along exterior edges perpendicular to applied load (chords), four 130 × 305 mm members spanning between chords (rafter), eight 88.9 × 241 mm members spanning between rafters (purlins), and 38.1 × 88.9 mm members spanning between rafters (subpurlins) spaced at 0.61 m on center. During testing, the diaphragm was loaded in a nonreversed cyclic protocol (using load control) at 23 points along one chord (simulating a distributed load) and restrained at the corners of the opposite chord.

For the finite element model, the modulus of elasticity of wood framing is approximated using the design values given in the National Design Specifications for Wood Construction: Supplement (AF&PA 2001). The modulus of elasticity, shear modulus, and effective shear thickness of sheathing are estimated using the design values given in the Plywood Design Specification (APA 1997). The chord splice stiffness is extrapolated from chord displacements measured during testing.

The force–displacement behavior of the sheathing-to-framing connections is described using the mathematical expression sug-

**Fig. 8.** Oriented spring pair model

\[
K_{11} = K_u \cos^2 \phi + K_v \sin^2 \phi
\]

\[
K_{12} = K_u \cos \phi \sin \phi - K_v \cos \phi \sin \phi
\]

\[
K_{22} = K_u \sin^2 \phi + K_v \cos^2 \phi
\]

\[
F = \begin{bmatrix}
P_u \cos \phi - P_v \sin \phi \\
P_u \sin \phi + P_v \cos \phi \\
-P_u \cos \phi + P_v \sin \phi \\
-P_u \sin \phi - P_v \cos \phi
\end{bmatrix}
\]
suggested by the APA–The Engineered Wood Association (APA 2001) for 12.7-mm-thick plywood and 3.76-mm-diameter \( \times \) 76.2-mm-long nail connections:

\[
P = 1.274A^{1/3.276}
\]  

(8)

In Fig. 9, the measured response and the finite element model response of the plywood diaphragm are shown. The diaphragm configuration and loading are depicted in the figure inset. For purposes of comparison, the finite element responses using both the nonoriented spring pair model and the oriented spring pair model are given.

The finite element model response generally agrees with the measured response. The stiffness of the finite element model is accurate during initial loading. As the applied force increases, the stiffness is slightly overestimated. This difference could be attributed to damage sustained during loading of the diaphragm. During testing, the applied load was halted at 127 kN (corresponding to a midspan displacement of 114 mm) when the hydraulic cylinders at the midspan reached maximum extension and observations suggested that failure was imminent. For this displacement, the finite element model using the oriented spring pair model over-predicts the force by 11% (the nonoriented spring pair model over-predicts by 18%). One possible cause for this overestimation may be the use of linear chord splice stiffnesses.

**Wood Shear Wall Analysis Using CASHEW**

The response of two 2.44 \( \times \) 2.44 m oriented strand board shear walls tested at the University of British Columbia (Durham 1998) are predicted using CASHEW. These experimental results have also previously been used to validate numerical models (Folz and Filiatrault 2001; He et al. 2001).

The two nominally identical shear walls use 9.53 mm thick OSB sheathing panels attached to framing using 2.67-mm-diameter \( \times \) 50.0 mm long spiral (threaded hardened-steel) nails. During testing, one wall was loaded monotonically and the other wall was loaded cyclically. The load was applied along the top side and the wall was restrained along the base.

A modified version of CASHEW that incorporates the oriented spring pair model is used. The original CASHEW computer program is modified by removing the connection spacing adjustment algorithm, by replacing the nonoriented spring pair stiffness matrix with the oriented spring pair stiffness matrix, and by adding an algorithm to extract the initial orientation of each sheathing-to-framing connection. The sheathing-to-framing connection force–displacement curve is described using a logarithmic expression with a linear softening branch:

\[
P = \begin{cases} 
(P_0 + r_1K_0\Delta)\left[1 - e^{-K_0\frac{\Delta}{P_0}}\right], & \text{if } \Delta \leq \Delta_{ult} \\
(P_0 + r_2K_0(\Delta - \Delta_{ult})), & \text{if } \Delta_{ult} < \Delta \leq \Delta_{fail} \\
0, & \text{if } \Delta > \Delta_{fail}
\end{cases}
\]

(9)

For reversed-cyclic behavior, CASHEW uses a modified form of the Stewart (1987) hysteresis model, which includes strength degradation, stiffness degradation, and pinching behavior. In Table 1, the parameters for Eq. (9) and the hysteresis model are given for 9.53-mm-thick oriented strand board and 2.67-mm-diameter \( \times \) 50.0 mm-long spiral (threaded hardened-steel) nail connections. The parameter values are determined experimentally (Durham 1998), except the \( r_4 \) value is set to 0.05, in accordance with previous studies (Rosowsky 2002).

In Fig. 10(a), comparison between the modified CASHEW response (using the oriented spring pair model) and the measured response is shown. The modified CASHEW response is fairly accurate during large amplitude loading, and less accurate during small amplitude loading. This response is reasonable because the hysteresis model does not consider the loss of strength during small amplitude loading. In Table 2, a summary of the cyclic response of the shear wall is given. The absolute difference between the measured value, as a percentage of the predicted value, is listed for the ultimate displacement \( \Delta_{ult} \), ultimate load \( F_{ult} \), and energy dissipation \( E_u \) (energy dissipation is accumulated after each time step). The ultimate displacement and load values for the modified CASHEW response are about 3% closer to the measured values than the original CASHEW response. Therefore, although either model may be considered an acceptable design tool, the primary advantage of the oriented spring pair model is that no scaling or adjustment is required to compensate for overestimation of sheathing-to-framing strength and stiffness, as is done in previous nonoriented spring pair models. For comparison, the ult-

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**Table 1. Force–Displacement Curve Parameters: 9.53 mm Oriented Strand Board/2.67 mm Spiral Nail**

<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>( P_0 ) (kN)</th>
<th>( P_1 ) (kN)</th>
<th>( \Delta_{ult} ) (mm)</th>
<th>( \alpha )</th>
<th>( \beta )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.561</td>
<td>0.061</td>
<td>-0.078</td>
<td>1.40</td>
<td>0.143</td>
<td>0.751</td>
<td>0.141</td>
<td>12.5</td>
<td>0.8</td>
<td>1.1</td>
</tr>
</tbody>
</table>
timate displacement and ultimate load values predicted by He et al. (2001) using a nonlinear finite element model (LIGHTFRAME3D) were 17 kN and 58 mm, respectively.

In these numerical examples, the results from laboratory testing of shear walls and diaphragms consisting of only framing, sheathing, and fasteners, need to be viewed in the proper perspective. The response of actual wood structures under seismic loading is influenced by additional factors, such as the contribution of nonstructural elements. In a recent study, interior and exterior finish materials, for example, significantly increased the lateral stiffness of a two-story single-family wood frame house during shake table testing (Filiatrault et al. 2002). Interestingly, the study also concluded that the effect of finish materials on the response of larger wood structures remains unclear because the relative contribution of finish materials could not be quantified. The analytical model presented in this paper, therefore, may also be used to clarify the contribution of nonstructural elements to the overall structural response.

Conclusions

Previous representations of sheathing-to-framing connections in wood shear walls and diaphragms are inadequate. Namely, the single spring model is viable for monotonic analysis but incapable of cyclic analysis, and the nonoriented spring pair model, while capable of both monotonic and cyclic analyses, generally overestimates the stiffness and strength of the connection. These are significant limitations.

The nonoriented spring pair model overestimates connection stiffness because the spring stiffness and force change relative to the displacement trajectory. As a consequence, connections located near sheathing panel corners have greater stiffness than connections located elsewhere, in proportion to the distance from the panel corner.

The new analytical model presented in this paper represents sheathing-to-framing connections using a spring pair oriented along the initial displacement trajectory. This trajectory corresponds to experimental observations of actual sheathing-to-framing-to-connection.

The potential application of the new analytical model is illustrated by two numerical examples. In the first example, using the oriented spring pair model in a finite element analysis of a plywood sheath provides a closer fit to measured data, compared to using the nonoriented spring pair model. In the second example, the new analytical model is successfully implemented into the specialized structural analysis program CASHEW. By implementing the oriented spring pair model, no scaling or adjustment is required. Further research is needed to demonstrate the usage of the analytical model in a dynamic time-history analysis, and to determine the relative contribution of nonstructural elements to the overall response of wood structures.

In summary, the oriented spring pair model provides three distinct advantages:

1. The analytical model is capable of both monotonic and cyclic analysis.
2. The analytical model may be implemented in a general purpose finite element program or in a specialized structural analysis program.
3. The analytical model is rigorous; it does not need to be scaled or adjusted.

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The writers gratefully acknowledge the assistance of Professor Helmut Prion and Professor Frank Lam, and Mr. Jianzhong Gu (Dept. of Wood Science, and Department of Civil Engineering, University of British Columbia) in furnishing data from testing of 2.44 × 2.44 m OSB shear walls.

References


Table 2. Cyclic Response of 2.44 × 2.44 m Oriented Strand Board Shear Wall

<table>
<thead>
<tr>
<th>Sheathing-to-framing connection element representation</th>
<th>$\Delta_{ult}$</th>
<th>$F_{ult}$</th>
<th>Energy absorbed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured response</td>
<td>66.0</td>
<td>20.4</td>
<td>2.59</td>
</tr>
<tr>
<td>Nonoriented spring pair</td>
<td>60.0</td>
<td>24.0</td>
<td>2.92</td>
</tr>
<tr>
<td>Adjusted nonoriented spring pair</td>
<td>60.0</td>
<td>22.0</td>
<td>2.68</td>
</tr>
<tr>
<td>Oriented spring pair</td>
<td>70.0</td>
<td>21.4</td>
<td>2.64</td>
</tr>
</tbody>
</table>

$^a$Difference = [measured/predicted].