MODELING LATERALLY LOADED LIGHT-FRAME BUILDINGS

By Richard J. Schmidt,1 Member, ASCE, and Russell C. Moody2

ABSTRACT: Current capabilities of accurately predicting the behavior of the various subassemblies of light-frame wood structures (walls, floors, and roofs) now meet (or exceed) the ability to characterize material properties. However, subassembly analysis techniques are not commonly incorporated in design practice. The deficiency lies in the inability to efficiently predict the interaction of the subassemblies which yields the behavior of the entire structure. The purpose of this study is to develop and validate a simple structural analysis model to predict the behavior of light-frame buildings under lateral load. The model is limited to the racking response of shear walls that are arranged in a rectangular fashion beneath a rigid ceiling/floor diaphragm. Nonlinear load-slip behavior of fasteners is utilized in an energy formulation to yield a three-degree-of-freedom representation of each story of the building. Predicted behavior from the analysis model agrees favorably with results from full-scale tests. The model provides a method for estimating the behavior of light-frame buildings under lateral loading and should lead to realistic shear wall strength and stiffness requirements for both residential and commercial buildings.

INTRODUCTION

Significant progress has been made in the development of models for analyzing the structural performance of light-frame subassemblies such as floor, wall, and roof systems. These models recognize the composite performance provided by the fasteners, framing, and sheathing. However, no rational procedure exists for analyzing the three-dimensional behavior of these systems as they are assembled in light-frame structures. Such a method is needed to take full advantage of the structural analysis models by better defining the actual loads that are transmitted to the components. The primary objective of the research reported here is to formulate a simple analysis technique to predict the nonlinear deformations of three-dimensional light-frame buildings subjected to lateral loads at and beyond normal design levels.

Analytical methods for predicting overall performance of light-frame structures are not well developed. Likewise, there have been many improvements in the level of knowledge on structural behavior and analysis in the last 40 years, the design procedure has changed only slightly. Many studies on major components such as floors, walls, and trusses have produced sophisticated analysis methods (Gromala and Wheat 1984). However, as noted by Polensak (1984), no method of analyzing, and thus designing, complete light-frame buildings has yet been developed.

This paper begins with a review of the state of the art in both performance and structural modeling of light-frame subassemblies. Efforts at full-scale modeling are then reviewed, as well as the results of full-scale structural

1Asst. Prof., Dept. of Civ. Engrg., Univ. of Wyoming, Laramie, WY 82071.

Note. Discussion open until June 1, 1989. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on April 6, 1988. This paper is part of the Journal of Structural Engineering, Vol. 115, No. 1, January, 1989.
evaluations. Finally, a model for predicting deformations and load distribution is proposed, and results are compared with data from the literature.

**BACKGROUND**

**Shear Response of Walls**

The in-plane stiffness of walls under shear loading is the most important factor influencing the distribution of lateral loads within the structure. Several techniques for analytically describing the nonlinear behavior of nailed racking walls have been proposed. Initial efforts at predicting the racking performance of sheathed wall sections resulted in empirical equations relating results of lateral nail tests to racking strength (Neisel 1958; Neisel and Guerrera 1956; Welsch 1963). Using an energy formulation, Tuomi and McCutcheon (1978) presented a theoretical approach for predicting racking strength of 8 ft × 8 ft panels based on lateral nail performance and found close agreement with experimental results for a variety of panel materials.

Foschi (1977) used an orthotropic finite element approach which is useful in predicting the behavior of diaphragms under shear loading. Easley et al. (1982) developed formulas for predicting the deformation of shear walls based on experience with light-gage metal buildings and also analyzed the walls with the general finite element system. Lin (1983) proposed a general finite element approach for diaphragm systems that recognizes the second-order effects of axial loads in the studs. Itani and Cheung (1983, 1984) and Itani and Obregon (1984) have also proposed a nonlinear finite element approach using a modified version of the program NONSAP.

A method of experimentally simulating nonlinear racking performance has been proposed by Naik et al. (1984). Linear methods were discussed by Itani et al. (1982). Castillo (1984) used a stepwise linear approximation to the fastener behavior with a finite element formulation to predict the stepwise linear racking behavior of walls. McCutcheon (1985) used nonlinear fastener behavior with an energy formulation to predict racking behavior. Wolfe (1983) applied the energy concept to walls sheathed with gypsum wallboard. More recently, Patton-Mallory and McCutcheon (1987) reported success in obtaining the correct connection properties by testing joints with sheets of plastic film between the connected pieces.

Gupta and Kuo (1982a) followed an approach similar to Easley et al. and McCutcheon in developing a closed-form solution for modeling the nonlinear deformation of shear walls. Gupta and Kuo (1985c) supplemented their analytical method to include uplift of the loaded end of the wall and later (Gupta and Kuo 1985b) considered uplift of the sole plate to more closely match results of full-scale tests of a complete structure.

Techniques for wall analysis have also been proposed by Burgess (1976), Kamiya (1981), Takino (1978), and Walker (1979). The performance of let-in braces was evaluated by Tuomi and Gromala (1977), and a technique for predicting strength was proposed. Hirashima et al. (1981) also proposed an analysis method for walls with diagonal bracing. The performance of walls sheathed on both sides was reported by Patton-Mallory et al. (1984).

**Full-Structure Modeling**

Analysis procedures for predicting force distributions within light-frame structures date back to 1948 (Whittemore et al. 1948). Since then, Tuomi
and McCutcheon (1974) showed that a simplistic distribution of forces was reasonable. Sugiyama et al. (1976) suggested that several elements neglected in simplistic analysis contribute significantly to overall structural stiffness. Noguchi and Sugiyama (1976, 1977) related wall racking stiffness to full-structure performance.

Itani et al. (1982) presented a technique for modeling shear walls with diagonal springs to determine force distribution. Subsequent work (Itani et al. 1982) has examined this approach further and determined that results are quite sensitive to boundary conditions. Analysis procedures for the torsional response of light-frame structures under lateral loading have been proposed by Hirashima et al. (1981) and Cziesielski (1982). Techniques have also been developed by Naik et al. (1984) to model behavior under seismic loading.

Elias and Fowler (1979) developed a linear approach to analysis of mobile homes under static loading. Simplified design techniques are recommended in England that recognize interior walls as resisting vertical loads but do not recognize their shear resistance (Burgess 1976; Trada 1977). On the other hand, procedures in Australia do recognize the shear resistance of interior partitions (Plywood Association of Australia, undated).

The analysis procedure developed by Gupta and Kuo (1985b) models the full structure performance under static lateral loading. They verified their work for shear walls using the Oregon State tests published by Itani and Cheung (1983) and the full structure test of Tuomi and McCutcheon (1974).

**Full-Scale Tests**

Dorey and Schriever (1957) evaluated a one-story 24 ft × 36 ft house under simulated wind and snow loads. The walls were constructed with 1 in. × 4 in. let-in diagonal wind braces and covered on the inside with 3/8-in. gypsum wallboard, but the exterior sheathing was purposely omitted. The experimental house evaluated by Hurst (1965) was evaluated during various stages of construction and dismantling. Wall movement under simulated wind loading was resisted primarily by the 3/8-in. plywood exterior wall sheathing, and it was difficult to quantify additional contributions of interior walls and gypsum wallboard sheathing.

Yokel et al. (1973) evaluated the performance of a conventional two-story house under simulated wind loads up to about 25 psf. Walls were constructed with gypsum wallboard on both faces and diagonal 1 in. × 4 in. wood braces in the corners. Yancey and Somes (1973) evaluated both the stiffness and strength of a housing unit typical of a factory-built module. Tuomi and McCutcheon (1974) evaluated the lateral resistance of a single-story light-frame structure during various stages of construction to determine the effects of window and door openings, interior gypsum board, and other structural components.

Sugiyama et al. (1976) evaluated a two-story house built following U.S. practices under simulated wind load. The house tested by Noguchi and Sugiyama (1976) was not typical of U.S. light-frame construction but does include extensive data on house stiffness. Tests of two 18 ft × 24 ft houses are reported by the Japanese Ministry of Construction (1975) and, although the construction differs somewhat from traditional U.S. construction, extensive data are given on deformations of both two-story structures. The house evaluated by Hirashima et al. (1981a) was small, with dimensions of about 203
12 ft × 18 ft. The house was sheathed with plywood, and deformation information is given for loading in two directions.

Extensive testing is being conducted at the James Cook Cyclone Structural Testing Station in Townsville, Australia, to evaluate light-frame wood structures under wind loading (Boughton 1982a, 1982b, 1984; Reardon 1985). Four houses have been evaluated, and a fifth test is underway. Two of those completed were new wood-frame units, and data for a simple structure, called the Tongan house, has been published (Boughton 1984; Reardon 1985a, 1985b).

In addition to the Australian work, research on both static and dynamic behavior of wood structures is underway at both the University of Tokyo and the Forestry and Forest Products Institute, Tsukuba, Japan. Several other full-scale structures have been tested under lateral loading, but results are not yet published.

**Analytical Approach**

The model developed in this study, called RACK3D, predicts the nonlinear response to lateral loading of three-dimensional light-frame structures, in which shear walls are the primary load-carrying mechanism. The model extends previous work in which the observed behavior of a sheathing panel in a wood-frame shear wall (Tuomi and McCutcheon 1974) is combined with nonlinear load/slip curves for fasteners (Foschi 1977; McCutcheon 1985). Individual shear walls consist of panels and are arranged beneath a rigid diaphragm, which is subjected to lateral loads. The rigid diaphragm approximates the behavior of the floor system or roof system that separates various levels of shear walls. Translation and rotation of the diaphragm acts to distribute the loads to the shear walls as racking forces. Equilibrium iterations are performed to remove the residual load caused by the nonlinear response of the walls.

**Panel Behavior**

As shown in Fig. 1(a), the basic panel is composed of a perimeter wood-stud frame, a single sheet of sheathing, and nails. The basic panel may also contain interior studs and nails as appropriate. The vertical studs are assumed to be rigid with respect to bending and are pinned at their ends. When the frame is loaded with the racking force \( R \), it distorts into a parallelogram with the top and bottom plates remaining horizontal [Fig. 1(b)]. Under the action of the racking force, the nails connecting the sheathing to the frame distort, and the sheathing develops in-plane shear deformation. The total deformation of the frame \( \Delta_{f} \) is thus the shear deformation of the sheathing \( \Delta_{s} \), plus the deformation of the frame relative to the sheathing \( \Delta_{w} \).

Considering the sheathing to be a linearly elastic, edge-loaded plate, the shear deformation in the sheathing is derived from basic strength of materials as

\[
\Delta_{s} = \frac{RH}{GlL} \tag{1}
\]

in which \( \Delta_{s} = \) shear deformation of the sheathing; \( R = \) racking force; \( H = \) the panel height; \( G = \) the shear modulus of the sheathing; \( t = \) the sheathing
FIG. 1. (a) Basic Geometry of Basic Panel; (b) Distortion under Load of Basic Panel

thickness; and \( L \) = the panel length. The sheathing acts as a spring in series with the nails, so it carries the full racking force \( R \).

The composite response of the nails was determined by McCutcheon (1985) by equating the work of the external racking force that causes nail deformation to the internal energy in the nails. The work \( E \) of the external force causing nail distortion is the area under the \( R \) vs \( \Delta_n \) curve or

\[
E = \int R d\Delta_n
\]

from which

\[
R = \frac{dE}{d\Delta_n}
\]

If a relationship between nail force \( P_n \) and nail distortion \( \delta \), can be written, internal energy \( I \) is found from

\[
I = \sum_{n=1}^{m} P_n d\delta_n
\]

where \( m \) = the number of nails in the panel. Equating \( E \) with \( I \) gives

\[
R = \frac{dI}{d\Delta_n} = \sum_{n=1}^{m} \frac{d}{d\Delta_n} P_n d\delta_n
\]

In order to evaluate this expression, nail distortion \( \delta \), must be related to the component of frame distortion \( \Delta_N \).

Experimental studies indicate that the corner nails in the panel deform approximately along diagonal lines and that distortion of nails along an edge
can be interpolated from those at the comers [see Fig. 1(b)]. The distortion of a corner nail is

\[ d = \frac{1}{2} \Delta_n \sin \alpha \]  

where \( \alpha \) = the angle that the diagonal makes with a vertical edge [see Fig. 1(a)]. Assuming that nails are spaced symmetrically around the perimeter of the panel, the horizontal and vertical components of nail direction, \( \delta_{xn} \) and \( \delta_{yn} \), are

\[ \delta_{xn} = \pm d \sin \alpha \]  
\[ \delta_{yn} = \pm \left( \frac{j}{n_x} - 1 \right) d \cos \alpha \]  

along the top and bottom edges, and

\[ \delta_{xn} = \pm \left( \frac{j}{n_y} - 1 \right) d \sin \alpha \]  
\[ \delta_{yn} = \pm d \cos \alpha \]  

along the left and right edges, in which \( n_x \) and \( n_y \) = the number of nail spaces in the horizontal and vertical directions; \( i = \) a horizontal nail index: \( 1 \leq i \leq n_x \); and \( j = \) a vertical nail index: \( 1 \leq j \leq n_y \) (corner nails are counted just once). Total nail distortion is then

\[ \delta_n = (\delta_{xn}^2 + \delta_{yn}^2)^{1/2} \]  

Substitution of Eqs. 6 and 7 into Eq. 8 yields

\[ \delta_n = Q_n \Delta_n \]  

where

\[ Q_n = \frac{1}{2} \sin \alpha \left[ \sin^2 \alpha + \left( \frac{j}{n_y} - 1 \right)^2 \cos^2 \alpha \right]^{1/2} \]  

along the top and bottom edges and

\[ Q_n = \frac{1}{2} \sin \alpha \left[ \left( \frac{i}{n_x} - 1 \right)^2 \sin^2 \alpha + \cos^2 \alpha \right]^{1/2} \]  

along the left and right edges.

A functional relationship for the nail load-slip behavior is desirable. An exponential function was proposed by Foschi (1977) and provides some flexibility in matching test data. The function has the form:

\[ P_n = (A + B \delta_n) \left[ 1 - \exp \left( \frac{-C \delta_n}{A} \right) \right] \]  

where \( A, B, \) and \( C \) = constants that are matched to experimental load-slip data. A geometric representation of these constants is shown in Fig. 2. Using this exponential curve, the initial energy of nail distortion is
This load-deformation relationship for the panel has the same general form as the load-slip function for the nails.

In general, a panel will have some pattern of interior nails which are also assumed to be symmetrically located. Distortions of the interior nails are linearly interpolated from those along the edges. Consider nail $g$ in Fig. 3. The components of nail distortion at $g$ are

$$
\delta_{xg} = r_x \delta_x \\
\delta_{yg} = r_y \delta_y
$$

where $\delta_x$ and $\delta_y$ are the $x$- and $y$-components of distortion at the top left corner of the panel; and $r_x$ and $r_y$ are the length and height reduction factors. For these nails, the factor $Q_n$ is adjusted such that

$$
Q_n = \frac{1}{2} \sin \alpha \left[ r_{xg} \left( \frac{2}{r_{yg}} - 1 \right)^2 \sin^2 \alpha + r_{yg}^2 \cos^2 \alpha \right]^{1/2}
$$

for any vertical line of nails with $n$, nail spaces. The index $i$ varies from 0- $n$, for vertical interior studs.

A simple iteration procedure is used to enforce continuity between the two components of panel deformation, $\Delta_x$ and $\Delta_y$. For a specified value of total panel deformation $\Delta_r$, $\Delta_x$ is estimated, and $R$ is computed from Eq. 13. With that racking force, $\Delta_y$ is computed from Eq. 1, and the two deformation components are summed:
If $\bar{\Delta}_r$ differs from $\Delta_r$ by more than an acceptable tolerance, a new estimate of $\Delta_N$ is computed from

$$\Delta_N \leftarrow \frac{\Delta_N \bar{\Delta}_r}{\Delta_r} \quad \text{.................. (17)}$$

and the computations are repeated.

**Wall Behavior**

Evaluating behavior of a wall is simply a matter of summing the effects of the individual panels contained in the wall. As all panels must be the same height, all will undergo the same deformation as the wall. Locations that contain door or window openings are open over the full height of the wall; partial panels above and below openings are not included.

Any restraint to panel deformation from frictional and bearing contact with the floor, ceiling, or adjacent panels is ignored. Also, walls carry only racking forces; restraint to out-of-plane bending or torsion is not considered.

**Rigid Diaphragm Behavior**

The rigid diaphragm serves to combine the shear walls in each story into a three-degree-of-freedom (DOF) system. The diaphragm lies in the $x$-$y$ plane, and all walls are oriented parallel to either the $x$-$z$ plane or the $y$-$z$ plane (see Fig. 4). Each wall is attached to the diaphragm at a single point that lies in the $x$-$y$ plane midway between the ends of the wall. Racking forces are transmitted through this attachment point from the diaphragm to the wall. External loads are applied anywhere in the plane of the diaphragm and, since
the diaphragm is rigid, can be transformed to an equivalent force-couple system acting at the origin $o$. The displacement components of the origin $(u_o, v_o, \theta_o)$ define the DOF for the story.

In the following development, only a single story is considered. The procedure for multistory buildings requires a simple assembly of the individual story equations. Given displacements at the origin, the imposed displacements at any wall $i$, with attachment point coordinates $(x_i, y_i)$ are

$$u_i = u_o + x_i (\cos \theta_o - 1) - y_i \sin \theta_o$$  \hspace{1cm} (18a)

$$v_i = v_o + y_i (\cos \theta_o - 1) + x_i \sin \theta_o$$  \hspace{1cm} (18b)

$$\theta_i = \theta_o$$  \hspace{1cm} (18c)

Assuming rotations are small, Eq. 18 can be recast in matrix form:

$$\{U_i\} = [\lambda_i] \{U_o\}$$  \hspace{1cm} (19a)

where:

$$\{U_i\} = \begin{bmatrix} u_i \\ v_i \\ \theta_i \end{bmatrix}$$  \hspace{1cm} (19b)

$$[\lambda_i] = \begin{bmatrix} 1 & 0 & -y_i \\ 0 & 1 & x_i \\ 0 & 0 & 1 \end{bmatrix}$$  \hspace{1cm} (19c)

and

$$\{U_o\} = \begin{bmatrix} u_o \\ v_o \\ \theta_o \end{bmatrix}$$  \hspace{1cm} (19d)
The contribution of forces in wall $i$ to the total force on the diaphragm at
the origin is

$$\{F_0\} = \{\lambda_i\}^T \{F_i\} \quad \text{.................................................. (20a)}$$

where

$$\{F_0\} = \begin{bmatrix} F_{x0} \\ F_{y0} \\ M_w \end{bmatrix} \quad \text{.................................................. (20b)}$$

and

$$\{F_i\} = \begin{bmatrix} F_{xi} \\ F_{yi} \\ 0 \end{bmatrix} \quad \text{.................................................. (20c)}$$

Since each wall lies parallel to a coordinate axis and carries only racking
force, only one nonzero entry will exist in $\{F_i\}$.

Racking force in a wall can be related to racking displacement by

$$[K_i][U_i] = \{F_i\} \quad \text{.................................................. (21a)}$$

where

$$[K_i] = \begin{bmatrix} k_x & 0 & 0 \\ 0 & k_y & 0 \\ 0 & 0 & 0 \end{bmatrix} \quad \text{.................................................. (21 b)}$$

in which either $k_x$ or $k_y = \text{the wall racking stiffness}$. For walls oriented in
the $x$-direction, $k_y = 0$; for walls oriented in the $y$-direction, $k_x = 0$. Using
the conventional assembly procedure, the total system stiffness is the sum
of the contribution from each wall:

$$[K_0] = \sum_{i=1}^{w} [\lambda_i]^T [K_i] [\lambda_i] \quad \text{.................................................. (22)}$$

where $w = \text{the number of walls}$. The resulting equilibrium equation for the
system:

$$[K_0][U_0] = \{F_0\} \quad \text{.................................................. (23)}$$

is nonlinear and must be satisfied through iteration. Because the load-slip
relationship of Eq. 11 is based on total displacement, the computations in­
volve secant stiffness rather than tangent stiffness.

**Fastener Performance**

The racking model used in the RACK3D analysis procedure requires ma­
terial properties for the sheathing and an accurate characterization of the
load-slip behavior of the fasteners. As previously noted, the three-parameter
exponential curve proposed by Foschi (1977) will be considered.

Since the actual load-slip behavior for the materials used in the full-scale
tests is unknown, behavior is estimated based on similar data in the litera­
ture. The important range of slip for nails in shear walls is well below 0.1
in. Most conventional walls will be deformed well past a serviceability limit
state when the individual nails have slipped only 0.02–0.03 in. Thus, the nonlinear behavior of the nails up to these slip levels has been characterized. The experimental data used to generate the fastener load/slip curves for this study are taken from Gromala’s tests (1985) with adjustments as suggested by Patton-Mallory and McCutcheon (1987) and Aune and Patton-Mallory (1986).

MODEL VALIDATION

Of the full-scale tests cited, only those by Tuomi and McCutcheon (1974) (the FPL 234 house), Sugiyama (1976), Hirashima (1981a), and Boughton (1984) (the Tonga house) are typical of the US.-type construction. Of these, only the FPL 234 house and the Tongan house will be used to validate the RACK3D model. Detailed comparisons of the model with the remaining full-scale tests are reported elsewhere (Moody and Schmidt 1988).

FPL 234 House

The house test reported in Tuomi and McCutcheon (1974) was a relatively simple 16 ft × 24 ft single-story house with plywood exterior sheathing and gypsum wallboard interior sheathing on the walls. The roof consisted of trusses on two-foot centers covered with plywood sheathing. Also, the ceiling was sheathed with gypsum wallboard. Only two of the shear walls in the house were loaded, and each of these walls contained two windows and a door opening. These two walls were loaded with jacks at their tops and then loaded in the opposite direction to determine their behavior under load reversals. Results were presented in the form of load versus deformation curves for each wall. The house plan, loading, and model parameters are shown in Fig. 5.

![Diagram of FPL 234 House](image)

**FIG. 5.** FPL 234 House—Plan, Loading, and Model Parameters
Extensive information on the construction of the walls and the materials used is included in the test report; thus, accurate estimates of model parameters could be made. Fig. 6 shows the experimental and the predicted/load-deformation results for the structure. In this case, the predicted response using RACK3D is in excellent agreement with the experimental results. Since only symmetric loading was applied, as shown in Fig. 5, the full-scale test does not provide data with which to evaluate the ability of the analytical model to predict torsional behavior.

Tongan House

The Tongan house evaluated by Boughton and Reardon (1984) was a relatively simple structure representing a single-story 16 ft × 24 ft house made with 2 in. × 4 in. framing and plywood sheathing on the exterior walls. Two short, symmetric interior partitions were sheathed with hardboard. Trusses supporting the corrugated metal roofing were spaced on two-foot centers and located in line with the studs. The design load calculated for the long side of the structure was 6,780 lb applied along the eave line of the south wall. Results are presented in the form of wall stiffnesses for each of the two exterior walls and for the interior partitions (Reardon and Boughton 1985). Limited information on the construction details and no material property information are included in the published reports. Thus, estimates of model parameters are likely to be less accurate than for the FPL 234 house. Fig. 7 shows the plan view of the Tongan house, the loadings, and the assumed model parameters.

Experimental and predicted response to both the symmetric and eccentric loading is given in Tables 1 and 2. Since the roof trusses were sheathed with corrugated metal roofing and the ceiling was not sheathed, the diaphragm was quite flexible. As a result, under symmetric load, the interior partition walls carried significantly more load than the analytical model predicted. However, predicted wall deformation is within the range of experimental
values, which indicates that the model provides a good approximation of overall structure stiffness. Under the action of symmetric loading, the traverse (north and south) walls carried no racking force.

The eccentric load consisted of a 3,390-lb concentrated force applied at the west wall. In the experiment, 64% of the load was resisted by the west wall, while the model predicted that it would carry 68%. The flexibility of the ceiling/roof diaphragm is again evident by the disparity in load distribution to the other walls. Experimental deformation data and the load carried by the north and south walls are not in the published report (Reardon and Boughton 1985). Consequently, predicted stiffness could not be validated.

Considering that the sheathing and fastener properties were estimated and that the rigid diaphragm used in the model does not represent the

![Diagram of Tongan House with dimensions and notes on sheathing and load parameters.]

**FIG. 7. Tongan House—Plan, Loading, and Model Parameters**

<table>
<thead>
<tr>
<th>Wall</th>
<th>Load (lb)</th>
<th>Deformation (in.)</th>
<th>Load (lb)</th>
<th>Deformation (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>East</td>
<td>2,530</td>
<td>0.16</td>
<td>2,860</td>
<td>0.20</td>
</tr>
<tr>
<td>West</td>
<td>2,560</td>
<td>0.16</td>
<td>2,860</td>
<td>0.20</td>
</tr>
<tr>
<td>Partition</td>
<td>1,690</td>
<td>0.28</td>
<td>1,060</td>
<td>0.20</td>
</tr>
</tbody>
</table>

**TABLE 1. Response to Lateral Load at Tongan House: Symmetric Load Distribution and Deformations**

<table>
<thead>
<tr>
<th>Wall</th>
<th>Test load (lb)</th>
<th>Predicted load (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>East</td>
<td>136</td>
<td>339</td>
</tr>
<tr>
<td>West</td>
<td>2,170</td>
<td>2,305</td>
</tr>
<tr>
<td>Partition</td>
<td>1,084</td>
<td>746</td>
</tr>
</tbody>
</table>
actual full-scale test, the agreement between the experimental and predicted results are quite acceptable.

CONCLUSIONS

A structural analysis model was developed to predict both the deformation and load distribution behavior of light-frame structures under lateral loading. The model predicts the lateral translation and rotation of a rigid ceiling/floor diaphragm that is restrained by the nonlinear shear walls. A comparison of the predicted behavior of two residential-sized buildings reported in the literature reveals reasonable agreement with the actual performance in terms of lateral translations and rotations. Thus, the model can serve as a simple tool for estimating the lateral load behavior of light-frame buildings. The next phase of development will extend the model to include arbitrary wall orientations and an elastic diaphragm to represent floor and ceiling response.

NOTE

The Forest Products Laboratory which employs the second writer is maintained in cooperation with the University of Wisconsin.

APPENDIX I. UNITS CONVERSION FACTORS

<table>
<thead>
<tr>
<th>To convert</th>
<th>Multiply by</th>
</tr>
</thead>
<tbody>
<tr>
<td>inch (in.)</td>
<td>millimeter (mm)</td>
</tr>
<tr>
<td>feet (ft)</td>
<td>millimeter (mm)</td>
</tr>
<tr>
<td>pound force (lb)</td>
<td>newton (N)</td>
</tr>
<tr>
<td>pound force per inch (lb/in.)</td>
<td>newton/mm</td>
</tr>
<tr>
<td>pound force per square inch (psi)</td>
<td>kilopascal (kPa)</td>
</tr>
<tr>
<td>pound force per square foot (psf)</td>
<td>kilopascal (kPa)</td>
</tr>
</tbody>
</table>

APPENDIX II. REFERENCES


214


analysis and performance.” Proc., 1988 Int. Conf. on Timber Engineering, Se­
attle, Wash.
thing.” Tappi J., 39(9), 625–628.
of platform construction due to full-scale house test and ordinary racking test (No.
of platform construction due to full-scale house test and ordinary racking test (No.
formance of light-frame walls sheathed on two sides.” Res. Pap. FPL 448, U.S.
Department of Agriculture, Forest Service, Forest Products Laboratory, Madison,
Wis.
of walls sheathed on both sides.” For. Prod. J. 37(9), 27–32.
ASCE Workshop on Structural Wood Research, Milwaukee, Wis., 189-203.
Reardon, G. F. (1980). “Recommendations for the testing of roofs and walls to resist
high wind forces.” Tech. Rep. No. 5, James Cook Cyclone Testing Station,
Townsville, Queensland, Australia.
Div. Tech. Papers, Brisbane, Australia, 26(20), 1–6.
gan hurricane house.” Presented at Asia Pacific Symposium on Wind Engineering,
Roorkee, India.
Structural plywood wall bracing: design manual. (undated). Plywood Association
of Australia, Newstead, Queensland, Australia.
wood bearing wall in relation to the framing type.” Wood Res. (Japan), 64, 33–
48.
bracing, sheet materials and effect of loading rate.” Res. Pap. FPL 301. U.S.
Department of Agriculture, Forest Service, Forest Products Laboratory, Madison,
Wis.
simulated snow loads and wind loads.” Res. Pap. FPL 234. U.S. Department of
Agriculture, Forest Service, Forest Products Laboratory, Madison, Wis.
Jubilee Conference, Institute of Engineers, Perth, Australia.
J., 46(8), 458–465.
principles to structural design.” Building Materials and Standards Report BMS
light-frame walls.” Res. Pap. FPL 439, U.S. Department of Agriculture, Forest
Service, Forest Products Laboratory, Madison, Wis.
APPENDIX III. NOTATION

The following symbols are used in this paper:

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>A, B, C</td>
<td>constants used in exponential load-slip function;</td>
</tr>
<tr>
<td>d</td>
<td>corner nail distortion;</td>
</tr>
<tr>
<td>E</td>
<td>work of external racking force;</td>
</tr>
<tr>
<td>{F}</td>
<td>force vector;</td>
</tr>
<tr>
<td>G</td>
<td>shear modulus of sheathing;</td>
</tr>
<tr>
<td>H</td>
<td>panel height;</td>
</tr>
<tr>
<td>l</td>
<td>internal energy;</td>
</tr>
<tr>
<td>i, j</td>
<td>nail indices;</td>
</tr>
<tr>
<td>[K]</td>
<td>stiffness matrix;</td>
</tr>
<tr>
<td>k</td>
<td>wall racking stiffness;</td>
</tr>
<tr>
<td>L</td>
<td>panel length;</td>
</tr>
<tr>
<td>m</td>
<td>number of nails in panel;</td>
</tr>
<tr>
<td>n, n</td>
<td>number of nail spaces;</td>
</tr>
<tr>
<td>P</td>
<td>nail force;</td>
</tr>
<tr>
<td>Q</td>
<td>panel geometry factor;</td>
</tr>
<tr>
<td>R</td>
<td>racking force;</td>
</tr>
<tr>
<td>r, r</td>
<td>reduction factors;</td>
</tr>
<tr>
<td>t</td>
<td>sheathing thickness;</td>
</tr>
<tr>
<td>{U}</td>
<td>displacement vector;</td>
</tr>
<tr>
<td>u, v</td>
<td>displacement components;</td>
</tr>
<tr>
<td>x, y</td>
<td>geometric coordinates;</td>
</tr>
<tr>
<td>α</td>
<td>angle made by diagonal with vertical edge;</td>
</tr>
<tr>
<td>Δr, Δs, Δv</td>
<td>components of panel deformation;</td>
</tr>
<tr>
<td>δn, δm</td>
<td>nail distortions;</td>
</tr>
<tr>
<td>θ</td>
<td>rotational displacement component; and</td>
</tr>
<tr>
<td>[λ]</td>
<td>geometric transformation matrix.</td>
</tr>
</tbody>
</table>