NON-LINEAR ANALYSIS OF LIGHT-FRAMED WOOD BUILDINGS

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SUMMARY

The paper presents a nonlinear analysis procedure that is effective for buildings that utilize sheathed stud walls as the primary elements in the lateral load resisting system. The load versus deformation relationships of the walls determine the earthquake response of wood buildings, and these relationships are governed primarily by the load versus slip relationships of the sheathing-to-frame connectors. An analytical procedure that calculates the response of timber walls is presented. The technique calculates wall behavior in the nonlinear range by equating the work done by the external lateral loads to the internal energy developed during deformation of the connectors. The procedure can be applied to walls with any spacing of studs, and any configuration of interior and exterior nailing. Analytical results are shown to compare favorably with experimental data and can provide information on sheathed walls of various dimensions and configurations when testing is not feasible or economical. The paper then presents a methodology for applying the nonlinear properties of individual walls to behavior of an entire wood-framed building. The approach, which is similar to that proposed for concrete buildings, incorporates the nonlinear and dynamic properties of the various building components and provides results for different levels of earthquake loading, as required by typical performance-based design criteria.

INTRODUCTION

Majority of the buildings in North America and other regions around the world are constructed with timber, yet the procedures that structural engineers use to design wood buildings are primitive when compared with the techniques used to design concrete, steel, or masonry buildings. The tremendous amount of damage to wood buildings during the Northridge earthquake revealed that the current methods used to design and analyze light-framed wood structures in regions of high seismic risk are inadequate. In addition, there is a trend in the structural engineering profession towards performance-based design, which requires analyses that are more sophisticated as well as the determination of structural response in the nonlinear range. It is thus evident that there needs to be an improvement in the techniques used to analyze wood buildings in seismic regions. This paper presents a procedure for the nonlinear analysis of wood buildings that utilize sheathed stud walls as primary lateral load resisting elements. The relationship between the slip of the sheathing-to-frame connectors and wall displacement is developed and a computer program is created to perform the computations required to obtain seismic response of a wall from the load versus slip relationships of the connectors. Once the properties of the walls are known, the performance of a building can be determined using a variety of analysis methods. This paper presents an example of building response calculated using a static nonlinear analysis, which is sometimes called a pushover analysis. With this methodology, the properties of the walls are assembled to create an analytical model that is subjected to progressively increasing displacements. The model is continually modified to account for the softening of the various components and the resulting redistribution of lateral loads. (Applied Technical Council, 1996, FEMA, 1997) The analysis provides an envelope to the nonlinear cyclicresponse of the building to earthquakes and provides a tool for evaluating building performance at various levels of displacement.

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NONLINEAR RESPONSE OF LIGHT-FRAMED SHEATHED WALLS

Shear walls in light-frame timber construction typically consist of a wood frame made up of studs, a top plate, and a sole plate; and applied sheathing panels of plywood, gypsum wallboard, oriented strand board, or other such material. The sheathing panels are attached to the frame with connectors such as nails, staples or screws. Holdowns are usually placed at the ends of the walls to resist overturning forces, and anchor bolts resist the shear force at the base. Figure 1 shows the layout of a typical wood shear wall.

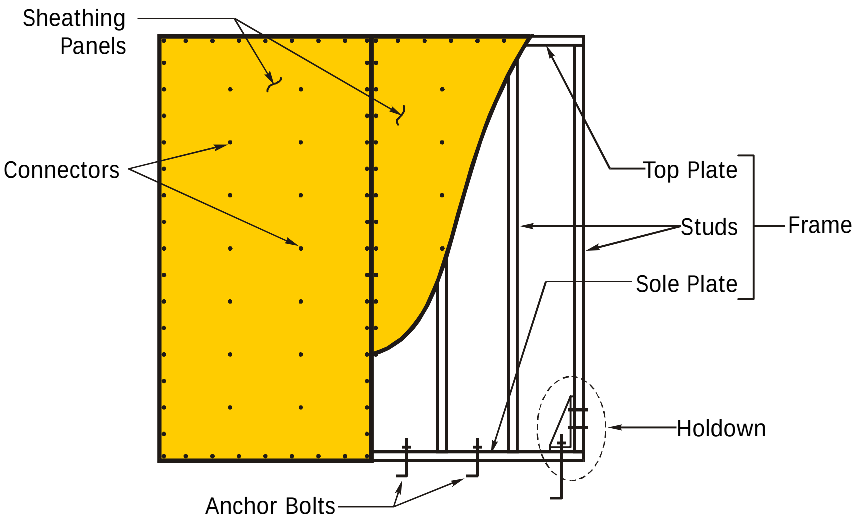


Figure 1 Components of a Typical Shear Wall in Light-Frame Building Construction

The lateral displacement of a sheathed stud wall is due to contributions from (i) the deformation of the sheathing material; (ii) the distortion, or slip, of the sheathing-to-frame connectors; (iii) the deformation and slip of the holdowns at the ends of the wall due to overturning moments; and (iv) the sliding of the sole plate relative to the foundation. The contribution of the last two items to total displacement is typically small for well-constructed walls with relatively stiff hold-downs. Thus, the first two parts act as springs in series so that the total displacement at the top of the wall, Δ_T, is given by:

$$\Delta_T = \Delta_S + \Delta_C \tag{1}$$

where and Δ_S is the displacement due to the shear deformation of the sheathing and Δ_C is the displacement due to connector slip. The shear deformation of the sheathing is equal to:

$$\Delta_S = \frac{FH}{GtL} \tag{2}$$

where H and L are the height and width of the sheathing panel, respectively; t is its thickness; and G is the shear modulus of the sheathing material.

When a sheathed stud wall is subjected to lateral displacements, the wood frame deforms into the shape of a parallelogram since it possesses little lateral resistance. However, the sheathing panels have a large in-plane stiffness and attempt to retain their rectangular shape. The connectors that attach the sheathing to the frame are thus subjected to distortion, or slip, due to the difference in deformation between the flexible frame and the stiff sheathing panels. Figure 2 shows a single sheathing panel and the phenomenon that occurs during lateral displacement. Since each panel in a wall acts independently, the response of a wall with many panels is the sum of the response of each individual panel (including the deformation of its connectors).

Laboratory tests have shown that the connectors at the corners of the frame slip approximately along the diagonals of the sheathing panel [Tuomi and McCutcheon, 1978]. Thus, the relationship between the corner connector slip, δ_c, and the displacement at the top of the wall, Δ_C, is:

$$\delta_c = \frac{1}{2}\Delta_C \sin\alpha \tag{3}$$

where α is the angle between the panel diagonal and the vertical edge as shown in Figure 2. The slip at any location on a panel can be obtained from the slip of the corner connectors. Then, using Equation (3), the slip for any connector can be calculated in terms of the displacement at the top of the wall as follows:

$$\delta_n = f(\delta_c) = Q_n \Delta_C \tag{4}$$

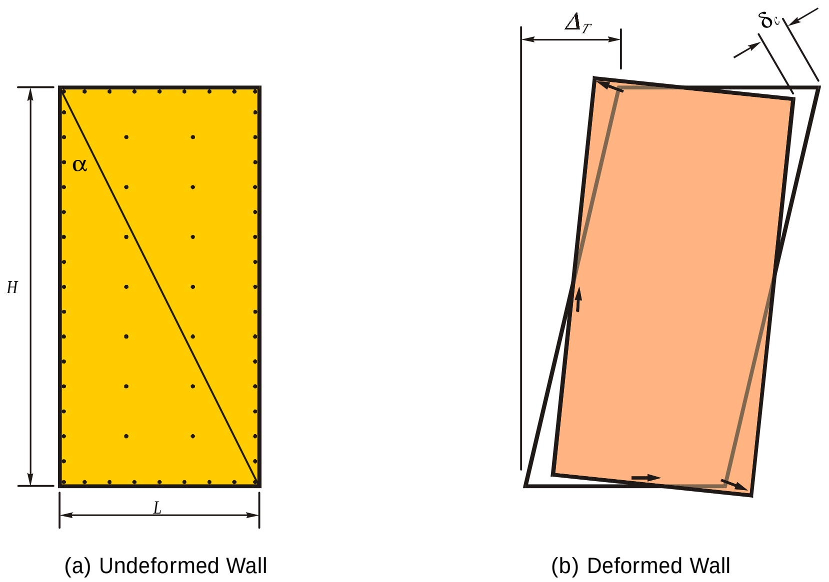


Figure 2 Deformation of a Sheathed Stud Wall

For connectors along the top and bottom edges of the panel Q_n is given by:

$$Q_n = \frac{1}{2}\sin\alpha\left[\sin^2\alpha + \left(2\frac{i}{n_x} - 1\right)^2 \cos^2\alpha\right]^{1/2} \tag{5}$$

and for connectors along the left and right sides of the panel Q_n is given by:

$$Q_n = \frac{1}{2}\sin\alpha\left[\left(2\frac{j}{n_y} - 1\right)^2 \sin^2\alpha + \cos^2\alpha\right]^{1/2} \tag{6}$$

The variables n_x and n_y are the number of connector spaces in the horizontal and vertical directions, respectively. The parameter i is a horizontal index used to identify the *ith* connection in the horizontal direction counting from the corner nails; and j is the vertical index used to identify the *jth* connection in the vertical direction counting from the corner nails. The slip of connectors located in the interior of a panel can be interpolated from the connectors along the edges. Therefore, for a vertical line of interior connectors:

$$Q_n = \frac{1}{2}\sin\alpha\left[r_{yn}^2\left(2\frac{j}{n_y} - 1\right)^2 \sin^2\alpha + r_{xn}^2 \cos^2\alpha\right]^{1/2} \tag{7}$$

where r_{xn} and r_{yn} are the length and height reduction factors that are a ratio of the interior connector layout to the exterior connector layout.

To determine the response of the wall in the nonlinear range, the external work done by the lateral force is equated to the internal energy created by the slip of the nails. The external work, W_e, done by the external lateral force, F, is the area under the force versus displacement curve. The internal energy or work, W_i, is obtained by adding the internal energy developed by each connector. This is computed by summing the area under each connector's load versus slip curve due to its slip at the given displacement. It then follows that the external lateral force for a given top displacement due to slip of the connectors is [Ekwueme and Hart, 1998]:

$$F = \frac{d}{d\Delta_C} \sum_{n=1}^{m} \int P_n d\delta_n \tag{8}$$

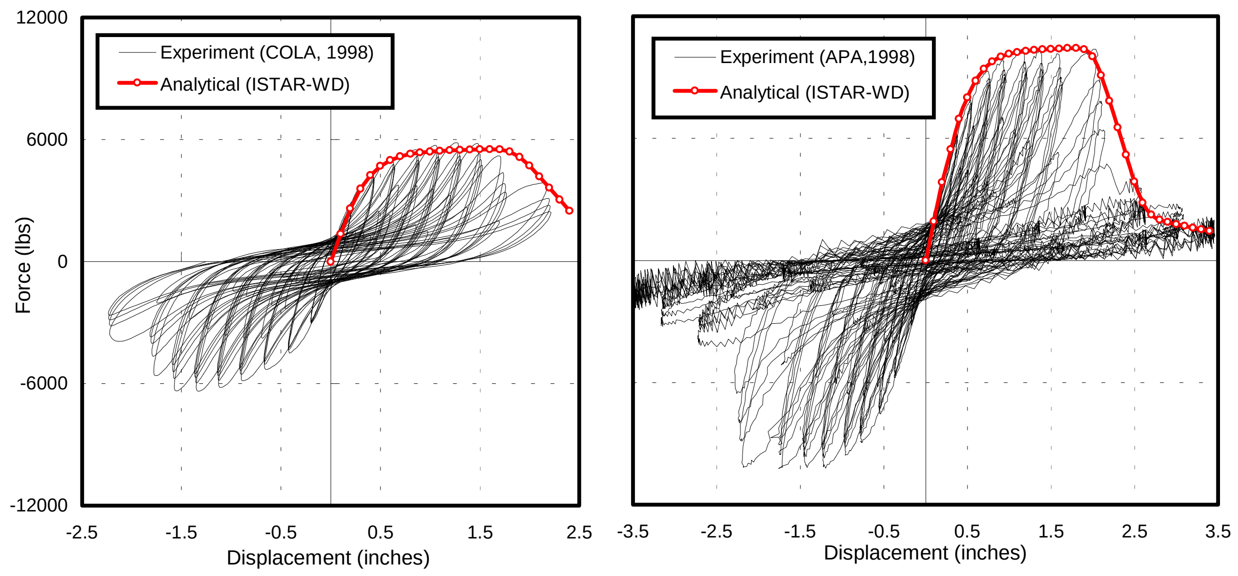
Thus, if the load versus slip relationship of the connectors is known, the external lateral force at a given wall displacement can be calculated, since the distortion of each connector, can be computed from Equations (3) through (7).

Equation (8) indicates that the wall displacement resulting from the distortion of the connectors depends on the load versus slip relationship of the connectors. Since this relationship is typically nonlinear, an iterative solution approach is required to determine what percentage of the total wall displacement is caused by the slip of the connectors and what percentage is due to the deformation of the sheathing. A computer program, ISTAR-WD [Ekwueme and Hart, 1998], was developed to perform the above computations to determine the response of sheathed stud walls to lateral loads. For a specified value of total displacement, Δ_T, the computer program calculates the displacement due to connector slip, Δ_C, and the external lateral force, F, is obtained by numerically integrating to satisfy Equation (8). Using the computed value of F, the displacement due to shear deformation of the panel is calculated from Equation (2). If the resulting total displacement differs from the target displacement by more than an acceptable tolerance (0.1% for ISTAR-WD) a new estimate of Δ_C is computed and the calculations are repeated until convergence is achieved.

VERIFICATION WITH EXPERIMENTAL DATA

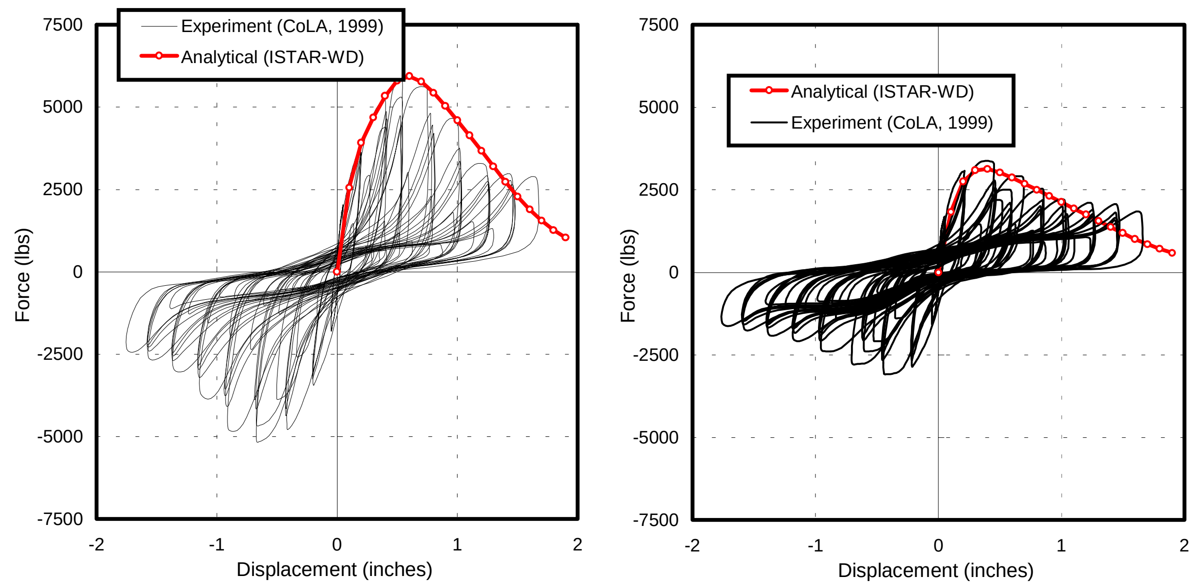
Results from computer analyses using ISTAR-WD were verified by comparisons with data from full-scale tests on light-framed shear walls. The comparisons allowed for a verification of the accuracy of the program's computational routines, and provided benchmark values for the input parameters, such as the variables that define connector load-slip relationships. Testing programs conducted by the Engineered Wood Association [Rose, 1998] and the City of Los Angeles [CoLA, 1999] at the University of California, Irvine provide abundant test data for verification. Tests were performed on 8-ft by 8-ft walls with various sheathing materials and connector configurations. Figure 3 and 4 show that analyses with ISTAR-WD are in excellent agreement with experimental data, and provide accurate envelopes of the cyclic response of walls sheathed with plywood and gypsum wall board (GWB).

In the experiments, sole plate sliding and hold-down slip and deformation contributed about 3-4% of the total lateral displacement. Therefore, ignoring their effect in the analysis was not significant. An exponential load-slip relationship, recommended by a number of researchers [Schmidt and Moody, 1989; White and Dolan, 1995] was used. The five-parameter mathematical representation is shown in figure 5. Table 1 provides the parameters used in the load-slip relationship for the various sheathing materials and nail sizes.



(a) (b)

Figure 3 Force versus Displacement Curves for 3/8" Plywood Walls with (a) 8d nails spaced at 6" and 12" (b) 8d nails spaced at 3" and 12".



(a) (b)

Figure 4 Force versus Displacement Curves for (a) 1/2" GWB with 1-5/8" Drywall Nails @ 7" and (b) 5/8" GWB with 1-7/8" Drywall Nails @ 4"

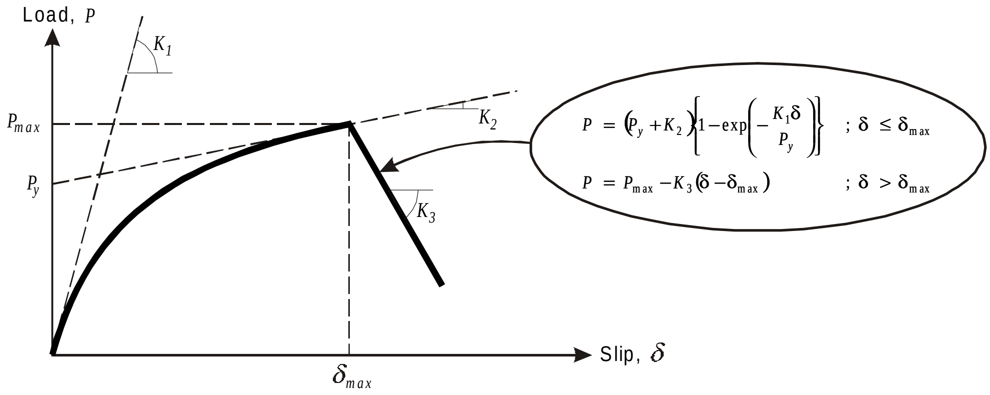


Figure 5 Exponential Load versus Slip Relationship for Sheathing-to-Frame Connectors

Table 1 Load versus- Slip Parameters for Sheathing-to-Frame Connectors

Connector and Sheathing Material	P_y (lbs)	P_{max} (lbs)	K_1 (lbs/in)	K_2 (lbs/in)	K_3 (lbs/in)
8d common nails on 3/8” plywood	269	272	9000	10	1500
1-5/8" drywall nails on 1/2” GWB	45	68	6000	300	150
1-7/8" drywall nails on 5/8” GWB	30	71	6000	300	200

ANALYSIS OF A WOOD-FRAMED APARTMENT BUILDING

Figure 6 shows the floor plans of typical wood-framed building with eight townhouse apartments. The properties of the walls are also shown. A static nonlinear analysis was performed using the appropriate wall properties. The analytical model was subjected to progressively increasing displacements using a triangular load distribution. At each level of displacement, the model was modified to account for the softening of the various components and the resulting redistribution of lateral loads. The floor and roof diaphragm were assumed to be stiff compared to the walls. Thus, forces were distributed to the walls in proportion to their stiffness at the given displacement. The assumption of a stiff diaphragm is confirmed by tests on wood buildings (Phillips, et. al., 1993). In addition, observations after earthquakes show that damage occurs much more frequently to walls than to the diaphragms of regularly shaped wood buildings. This implies that while the walls are displaced into the nonlinear range, the diaphragms remain essentially elastic and stiff when compared to the softened walls. Like most apartment and condominium-type structures, the building is symmetrical in both major directions. Torsional effects were therefore not considered in the analyses. For most detached residential houses, and other wood structures such as those characterized with “tuck-under parking”, the effects of torsion can be significant.

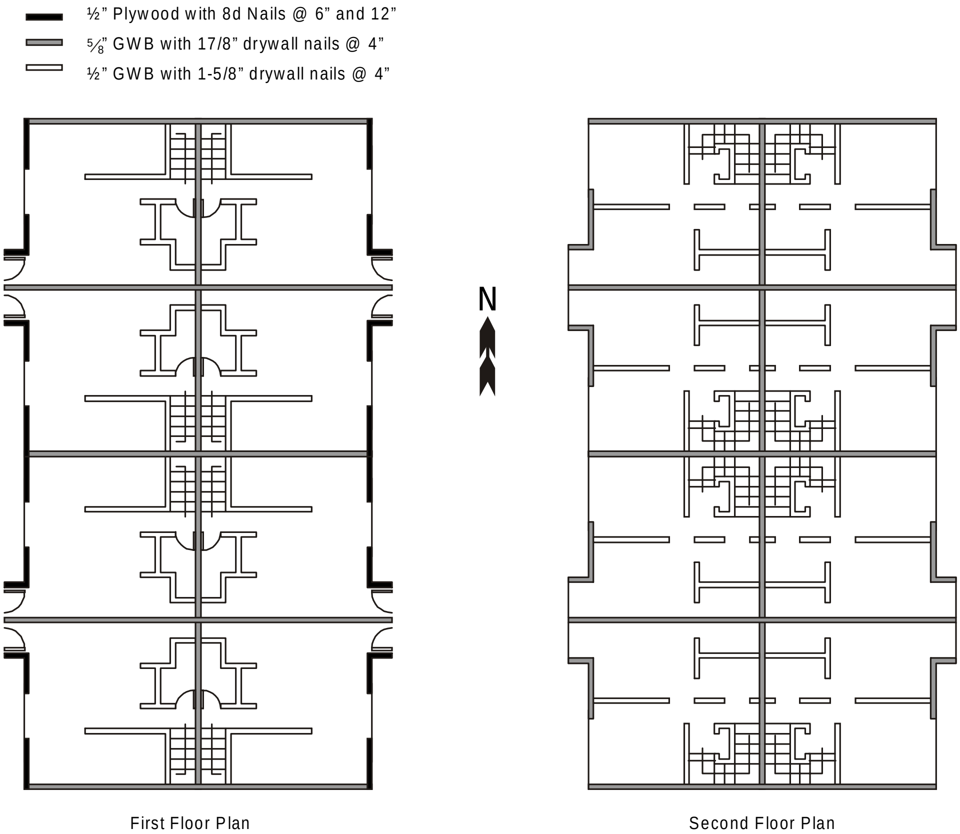


Figure 6 Typical Wood Townhouse Apartment Building

Figure 7 shows the results of the nonlinear static analysis in the North-South direction. As has been observed after earthquakes, the analysis shows that the damage first occurs to the interior gypsum wallboard in the first and second floors. Majority of the damage was incurred in the first floor, and the maximum base shear is attained at the strength limit state of the plywood in the first floor After attaining the maximum base shear, the strength of the building degrades rapidly, because of the large quantity of gypsum wallboard that provides lateral resistance. The analysis also showed that the building exhibits a soft-story phenomenon at large displacements, with a more flexible first floor and a relatively undamaged upper story.

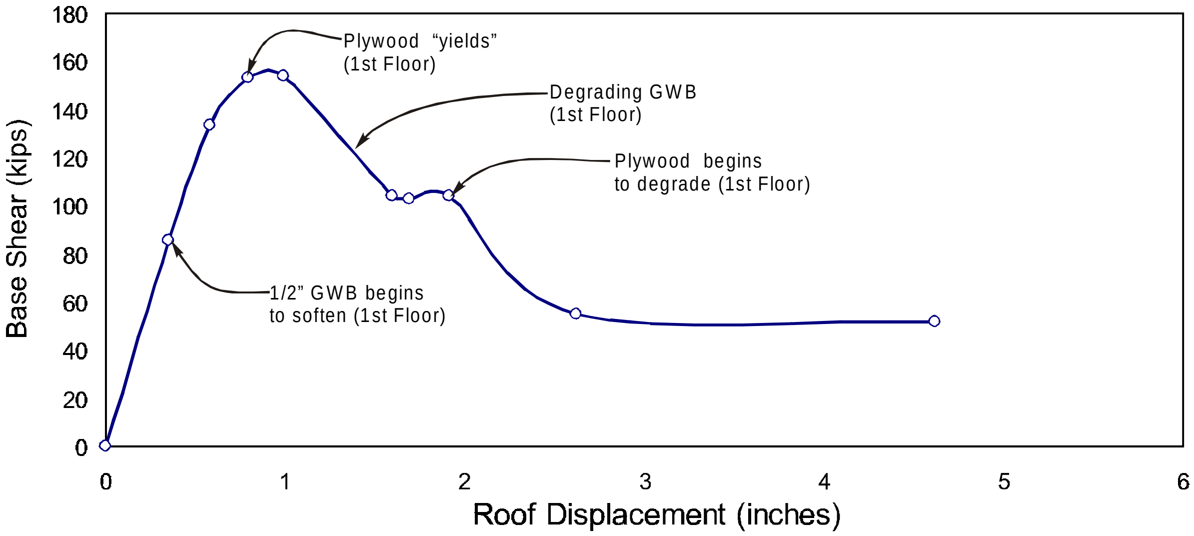


Figure 7 Lateral Response of Example Building in North-South Direction

CONCLUSIONS

The seismic response of a sheathed stud walls can be calculated from the load versus slip relationships of the connectors that attach the sheathing to the frame. Analyses with the computer program ISTAR-WD compare excellently with the results of full-scale testing of light -framed sheathed walls. Special consideration is needed for poorly constructed walls in which displacements due to holdown slip and deformation are a significant amount of total wall displacement.

The properties obtained from analyses of individual walls can be assembled to create an analytical model for determining the nonlinear response of a light framed wood building. A nonlinear static analysis or pushover analysis provides a tool for evaluating the damage in a wood building at various levels of displacement. Buildings with significant torsional response may require additional analyses. In addition, since wood buildings may have a soft first story at large displacements, the investigation of building performance may be improved by modifying the distribution of the lateral load to the floors of the building as the analysis progresses.

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