Damage-Based Seismic Reliability Concept for Woodframe Structures

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Abstract: In the United States the majority of modern residential and a portion of commercial structures are woodframe construction which typically consists of dimension lumber sheathed with 1.22×2.44 m (4×8 ft) OSB or plywood panels. This paper presents the results of a study on the development of a damage-based seismic reliability model for light-frame wood structures subject to earthquake load. It is presented in conceptual form with a simple example for a symmetric one-story woodframe building. The concept presented here provides the basic methodology to calibrate a seismic damage model and compute the structural reliability using a damage-based limit state function. The illustrative example necessitated a small experimental program and the use of existing software. The mechanistic damage model chosen for the example expresses damage as a linear combination of the maximum displacement during an earthquake simulation and the hysteretic energy dissipated by each shearwall within a structure. In order to make the model mechanistic, it was regressively calibrated based on the results of static and dynamic tests conducted on 2.44×2.44 m (8×8 ft) wood shearwalls. The fastener spacing on the perimeter of the shearwall sheathing panels was chosen as the design variable of interest, hence all damage model parameters were developed as a function of their spacing. Following calibration, a damaged-based limit state for reliability analysis is developed that enables one to calculate the structural reliability index provided against a prescribed damage level. The result for an example limit state is presented and compared to Federal Emergency Management Agency transient drift performance specifications for collapse prevention.

DOI: 10.1061/(ASCE)0733-9445(2005)131:4(668)

CE Database subject headings: Framed structures; Building frames; Earthquake loads; Damage; Structural reliability; Buildings, residential.

Introduction

Wood is the most widely used residential construction material in the United States. Woodframe buildings are relatively economical when considered on an individual basis, but their sheer number makes them critical to the infrastructure of the country and continued good seismic performance is integral to the overall economic welfare. The impact of poor seismic performance in the public sector (e.g., insurance companies, government, manufactures) as well as the private sector (e.g., insurance premiums, lack of insurance availability) would be significant. Over the years much of the light-frame wood structural design code development has been based on the results of monotonic and reversed-cyclic tests. While studies have been conducted using pseudodynamic tests (e.g., Dinehart and Shenton 1998; Kamiya 1988; Durham et al. 2001) and shake-table tests (e.g., Stewart 1997 Dolan and Madsen 1992a,b; Yamaguchi and Minowa 1998), the results have only just begun to find their way into design codes and guidelines. As more is learned about the nonlinear seismic behavior of woodframe structures, performance-based design philosophies similar to those under development for the design of steel and reinforced concrete structures are being developed for woodframe structures.

The remainder of this paper presents a new concept that is believed to fall under or within the paradigm of performance-based seismic design. Specifically, damage limiting criteria that may provide a prescribed reliability level against predefined damage levels are considered. In other words, it is reiterated here that one area for performance-based seismic design of light-frame wood structures to focus on is not performance with respect to collapse, but rather structural performance as it relates to damage, structural, or otherwise. This paper presents the concept of damage-based seismic reliability and the calibration of a preliminary (or prototype) damage model for calculation of damage-based seismic reliability of woodframe structures. This type of damage model has not been applied to woodframe structures, thus nuances and details have never been addressed and was one of the secondary objectives of the study presented here. The model focuses on wood shearwalls since state-of-the-art woodframe modeling (Folz and Filiatrault 2001b) techniques use an approach that is highly dependent on the hysteretic behavior of the shearwalls within the structure. The illustrative example presented later is by no means meant to be applicable to all woodframe buildings, but rather it is meant to demonstrate a basic application of the concept of damage-based seismic reliability. Further, it should be noted that this particular illustration of damage models and reliability has value in concept, but to be applied, i.e., for code calibration or development, will require consideration of finishes and nonstructural partitions.

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Note. Discussion open until September 1, 2005. Separate discussions must be submitted for individual papers. To extend the closing date by one month, a written request must be filed with the ASCE Managing Editor. The manuscript for this paper was submitted for review and possible publication on December 17, 2003; approved on October 11, 2004. This paper is part of the ***Journal of Structural Engineering***, Vol. 131, No. 4, April 1, 2005. ©ASCE, ISSN 0733-9445/2005/4-668–675/$25.00.

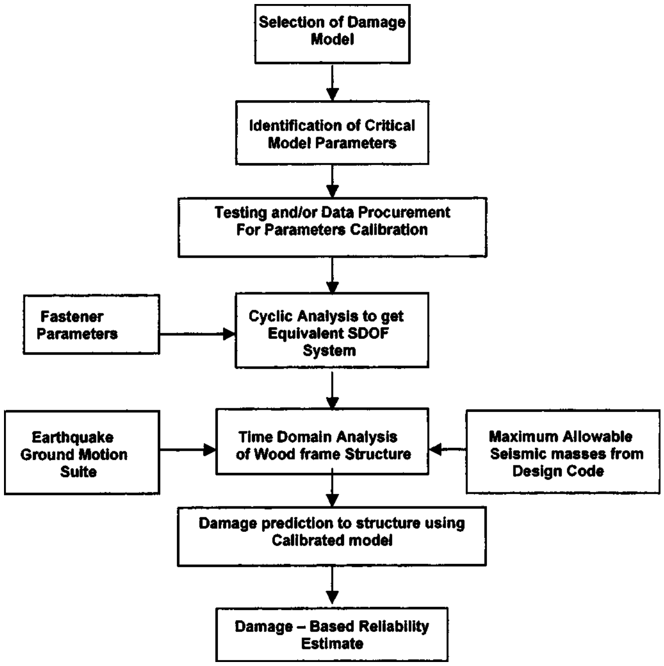

Fig. 1. Flowchart outlining the concept of damage-based seismic reliability for woodframe structures

Damage-Based Seismic Reliability Concept

To date, much of the seismic reliability of woodframe structures or more specifically wood shearwalls has been based upon deformation limit states (e.g., Rosowsky 2002; van de Lindt and Walz 2003) with the intent of eventually correlating these limit states to an acceptable level of damage. In fact, it has been said that limiting deformation is the cornerstone of performance-based seismic design (Filiatrault and Folz 2002). That approach is quite logical since basic mechanics provides methods for computing stresses from deformations. However, during strong earthquakes when moderate to severe damage occurs to structures, much of the basic linear mechanics breaks down and research engineers and scientists rely on more heuristic measures of performance adequacy ranging from experiment (i.e., observation) to experience. A concept is presented here that provides a mechanism to calibrate a seismic damage model and compute the seismic reliability of a woodframe structure based on one or more general behavioral (or performance) limit states. The methodology can be further generalized and is applicable to any type of system, but no effort to extensively generalize the approach was made as it was considered beyond the scope of this paper. Once the damage model is calibrated, the damaged-based reliability can be computed by (1) coupling the model with any nonlinear hysteretic model felt to be a good predictor of the dynamic behavior of the structure, and (2) utilizing a suite of appropriately scaled earthquake ground motions. Fig. 1 presents a flowchart outlining the overall concept for a woodframe structure, which is described below.

Beginning at the top of the flowchart in Fig. 1, initially one selects a damage model for use in the reliability analysis. Once the model is selected, the critical model parameters must be identified. The purpose of identifying critical parameters is to ensure that the damage model is made mechanistic for at least these parameters. This can be accomplished through regression about the mean or by making the model more generally stochastic [see, e.g., Park et al. (1985)] if sufficient test data is available. This may or may not require testing, but at the very least will require the procurement of appropriate data. Now, making the flowchart specific to woodframe structures at this point, each shearwall within the structure is analyzed cyclically and an equivalent nonlinear single degree of freedom (SDOF) hysteric oscillator can be defined. These nonlinear oscillators represent an assembly within the woodframe structural model (Folz and Filiatrault 2001b). Maximum allowable seismic masses for the structure can be determined from the appropriate design code if one is checking reliability levels inherent in that design code. At this point in the process, the damage model and structural model are calibrated and defined, respectively, essentially describing the resistance portion of the reliability problem. Existing (or determined) suites of ground acceleration records for the location of interest, i.e., site, provide the load portion of the reliability problem, provided they are scaled to the appropriate hazard level. The response of the structure, and each shearwall within the structure, is determined for each ground acceleration record in the suite of records. These responses are used as input to the calibrated seismic damage model and the damage for each shearwall within the structure can be estimated from the model. The total building damage can be estimated by combining the damage indices by weighting their contribution to the total damage based on the amount of hysteretic energy each wall dissipates.

Illustrative Damage Model

In order to illustrate the concept described above it is advantageous to select a straightforward seismic damage model. Consider the mechanistic seismic damage model originally developed for reinforced concrete (Park and Ang 1985) whose development was originally based on extensive test data for beams and columns (and beam-to-column connections) reported both in the US and Japan. Their approach was justified by stating that a structure will sustain damage due to both the maximum response and the effect of repeated cyclic-loading, which both occur during an earthquake. The simplicity of their approach may make it applicable to other materials and structural systems. The Park–Ang damage index, D, is expressed as

$$D = \frac{\Delta_m}{\Delta_u} + \frac{\psi}{F_{ey}\Delta_u}\int dE \qquad (1)$$

where Δ_m=maximum deformation during the earthquake; Δ_n=ultimate deformation (note that this is not the deformation that corresponded to ultimate capacity but rather the largest deformation reached prior to failure) under monotonic loading determined experimentally; F_{ey}=the yield force of the wall (which can be calculated as an equivalent yield force using a least-squares approach since wood shearwalls do not posses a distinct yield point); $\int dE$=incremental hysteretic energy absorbed by the wall during the earthquake; and ψ=calibration parameter for the desired damage-based limit-state.

Eq. (1) is a direct linear combination of the maximum deformation and the hysteretic energy absorbed during the motion of the Structure expressed as a fraction of their respective ultimate values. Only parameters Δ_m and $\int dE$ are obtained from the simulated loading history. All other parameters are predefined values and are essentially a function of the geometry and material properties of the component or structural system being modeled or analyzed. Damage to a structure is then characterized by the damage index, D, where the value of $D=1$ signifies failure or, at least,

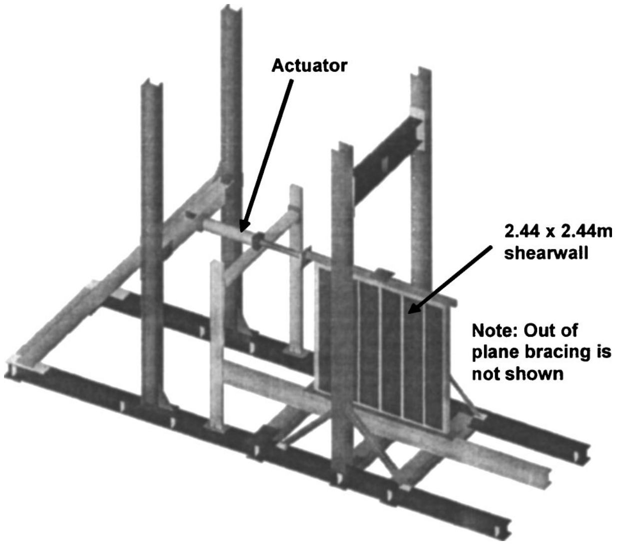

Fig. 2. Wood shearwall test setup for static and dynamic-displacement tests

that the predefined damage-based limit state has been reached.

Calibration of the Park-Ang (or any) damage model requires knowledge of certain structural properties, which were determined from shearwall testing under monotonic (MT) loading and dynamic-displacement (DD) loading. Dynamic-displacement testing is a displacement control dynamic test and will be explained in a later section of this paper.

Illustrative Damage Model Calibration

Recall from the previous introduction of the seismic damage model concept that one or more critical model parameters should be selected initially in order to make the model mechanistic. These critical model parameters will typically correspond to the critical design variables. For purposes of illustration, let us limit the damage model to wood shearwalls since current modeling techniques for woodframe structures are highly dependent on the hysteretic behavior of the shearwalls within the structure. In addition, the model calibrated here will only consider a single critical design parameter. The performance of wood shearwalls have been repeatedly shown to he significantly affected by the spacing of the fasteners around the perimeter (or exterior) of each sheathing panel. Thus exterior fastener spacing was selected as the design parameter upon which to make the example model mechanistic. In order to obtain the damage model parameters (see Eq. (1) several shearwall tests were performed. These included: (A) the results of monotonic shearwall tests to determine the Δ_u and F_{ey} values for walls having different exterior fastener spacing; and (2) dynamic shearwall tests to determine the Δ_m and $\int dE$ values. Fig. 2 shows a solid model of the test setup used; however, note that no out-of-plane bracing is shown. Sheathing panels on all shearwalls in this study were oriented vertically.

The term dynamic-displacement (DD) test is used here to mean a displacement control dynamic test. The load protocol for the DD test was determined by exciting a representative nonlinear hysteretic oscillator with the desired ground acceleration record numerically. A simple hysteretic SDOF oscillator model calibrated to existing shearwall data was used in the present study. The resulting deformation response time history was fed back through the actuator and the force response of the shearwall recorded. Ideally, shake table testing could be used, but neither the DD tests or shake table tests were the focus of this paper and are given limited attention. The use of a DD test can be further justified in that an example calibration of the Park–Ang damage model for woodframe structures is a fundamental example of an input-output (I–O) problem. In other words, the output is directly dependent on the input, and as long as the structure is deformed beyond the elastic range the model can be calibrated.

Limit State Selection

The selection of damage limit states was substantially more difficult in that it required (at this stage) some level of calibration with deformation-based limit states. In the present selection of limit state definitions, five (5) different criteria were used. These criteria were not meant to be absolute and representative of any particular damage level. Rather, they serve as preliminary limit states that were based on observations made during a series of reversed-cyclic wood shearwall tests not presented here. These included: A, either bottom corner nail pulling through the sheathing; B, the bottom corner of the sheathing displacing out-of-place more than 1.66 mm (1/16 in.); C, the sheathing displacing out-of-plane at mid-wall height more than 1.6 mm (1/6 in.); D, the bottom corner of the sheathing displacing out-of-place more than 3.2 mm (1/8 in.); and E, the sheathing displacing out-of-plane at mid-wall height more than 3.2 mm (1/8 in.). Note that all out-of-plane measurements were measured with respect to the framing. It should be further noted that the out-of-plane movement of the sheathing was generally observed as the fasteners worked their way out of the framing members during cycling. Although no direct quantitative correlation between failure(s) and this type of limit state has been shown, its observed consistency shows some promise. Its primary use in the present study was for expediency of illustration. Limit state A was selected as a potential serviceability overload limit state which, if reached, would most likely result in structural damage but probably not collapse. Limit states B, C, D, and E were selected as safety/collapse limit states since the shearwall will have lost significant load carrying capacity at this point. In order to provide some level of relativity to wall displacement, these limit states can be tied back to top-of-the-wall deformation-based reliability levels. This will be presented for limit state E and the results compared to existing Federal Emergency Management Agency (FEMA) collapse prevention limit states.

As mentioned, in order to calibrate the illustrative model, several static and several dynamic tests were needed. Fig. 3 shows the portion of the walls displacement created as a result of the 1949 Puget Sound ground motion for the shearwalls having 102 mm (4 in.) fastener spacing up until a selected limited state was reached. The responses for walls having other spacings of 76 mm (3 in.) and 152 mm (6 in.) are not presented here. The record was scaled in order to produce a significant transient drift to ensure the specimens were damaged. Without some reasonable level of damage the model would not have been able to be calibrated. Fig. 4 shows the resulting hysteresis for the wall obtained during the experiment using that displacement signal.

Regressing the Model Parameters on Fastener Spacing

Recall that in order to make a damage model mechanistic it should be a function of one or more design variables deemed

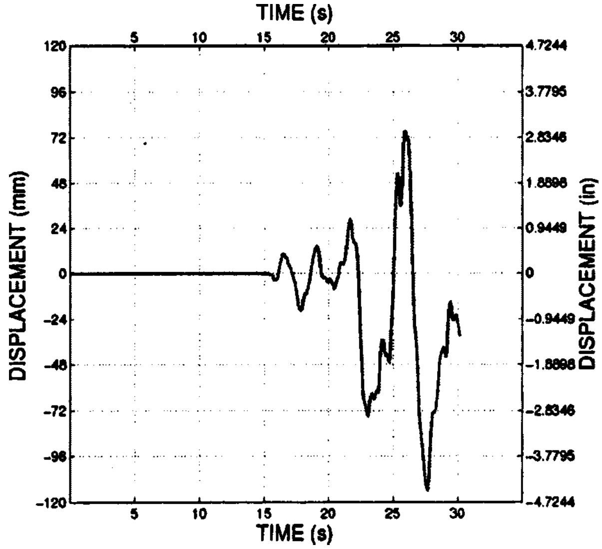

Fig. 3. Displacement of a wood shearwall until a predefined damage limit-state was reached

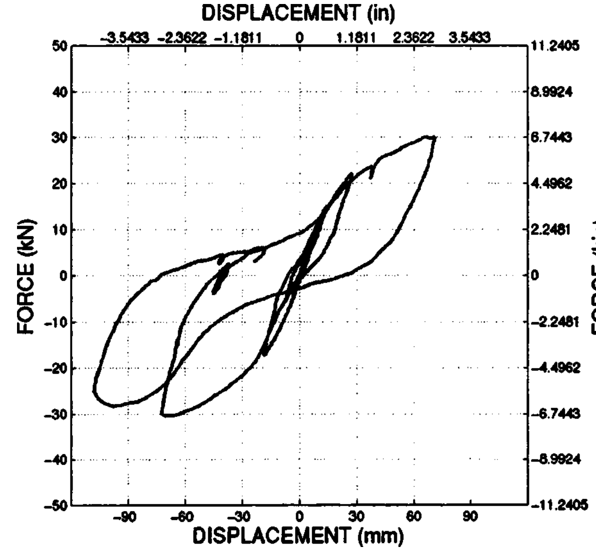

Fig. 4. Experimental hysteresis corresponding to the shearwall response

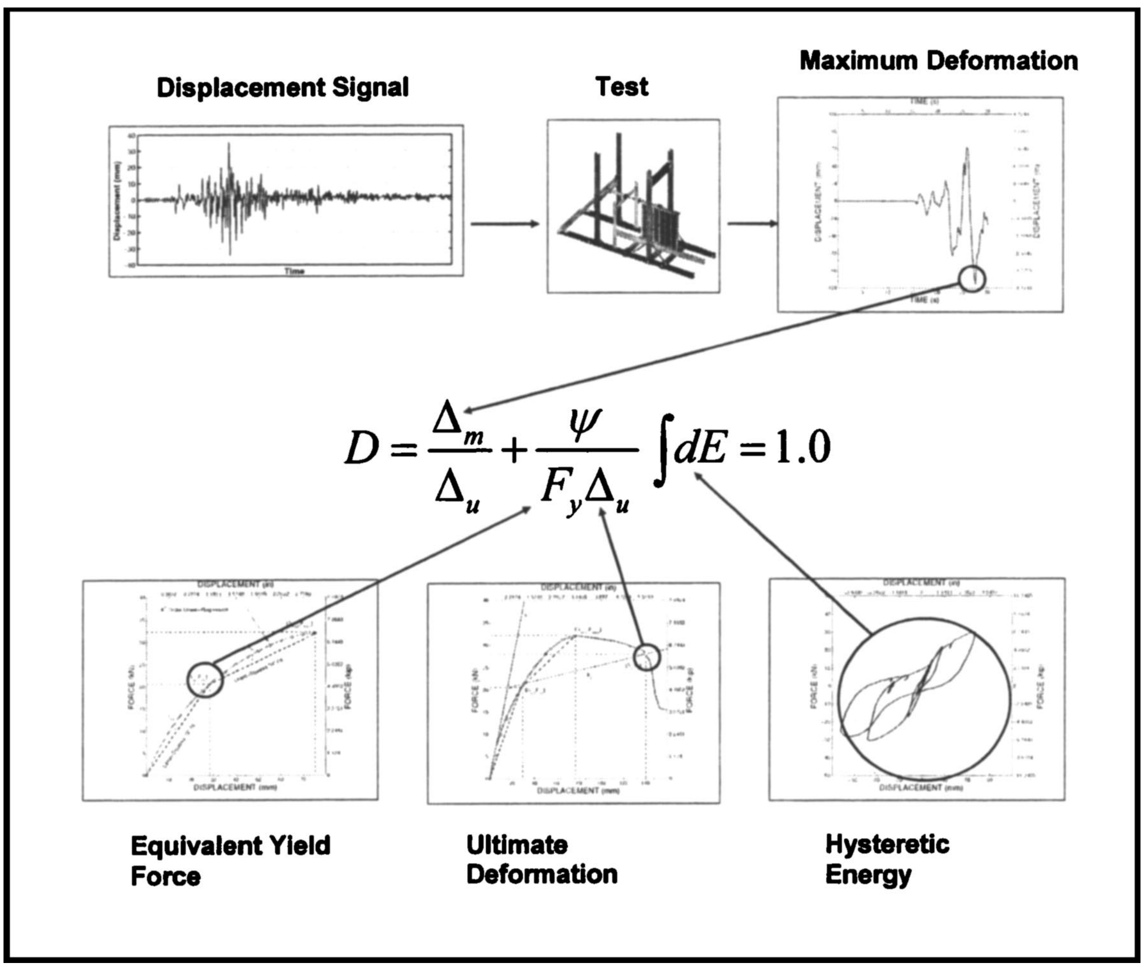

Fig. 5. Diagram showing the procedure for calculation of a single calibration parameter. Note that the procedure should be repeated for each test/nail spacing combination.

critical to the problem at hand. In the present example this was the fastener spacing for the perimeter nails. In order to calibrate the damage model using the two sets of test data and solve for the calibration parameter, ψ, one simply rearranges Eq. (1) as

$$\psi = \frac{F_{ey}(\Delta_u D - \Delta_m)}{\int dE} \tag{2}$$

Setting the damage index, D, equal to unity and integrating the hysteretic energy, $\int dE$, up to the point during the dynamic-displacement tests (as was presented in Fig. 3) at which the selected limit-state was reached, one may solve for the value of the calibration parameter. This also requires the maximum deformation, Δ_m, to be extracted from the dynamic-displacement test data. This model calibration procedure was obtained for each dynamic-displacement test, i.e., once for each earthquake record for a single fastener spacing. This was repeated for each of the three fastener spacings (76, 102, and 152 mm) for a total of six calibration parameters. Fig. 5 presents a diagram showing how one can go about determining the calibration parameter, ψ, for the damage model. Starting at the top left the control signal (in the present case displacement, but this could easily be replaced with acceleration for a shake table test) drives a single test of a wall (or structure). The absolute maximum displacement prior to the limit state being reached, Δ_m, is identified during the test. Recording the force as the wall is dynamically moved through the desired displacements (or the force-displacement relationship for a shake table test) the hysteretic energy dissipated by the wall was obtained. At the bottom left of the diagram an approach to determine an equivalent yield force and ultimate deformation from monotonic shearwall test data are depicted. Details of the approaches used in the example are available in van de Lindt et al. (2003). The expression for the damage index can then be set equal to unity and the value of the calibration parameter, ψ, readily determined. This procedure should be repeated for as many force-displacement records as are available for a single nail spacing (or any other design variable). The procedure is then repeated for each nail spacing. Specifically, two ψ values, i.e., one for each earthquake used to develop the displacement signal, were obtained for each walls nail spacing and are shown in Fig. 6. The two values were averaged for each wall type and a best-fit line used to calculate an expression for ψ as a function of nail spacing. Of course, the value of ψ changes for each limit state. So a new expression must be determined for each limit state. Table 1 presents the slope and y intercept for each linear expression of the calibration parameter ψ. Expressions for the equivalent yield strength, F_{ey}, and ultimate deformation, Δ_u, were also determined as a function of panel exterior nail spacing. Of course, these are not a function of limit state and can be expressed as

$$F_{ey} = -0.16s - 38.5 \tag{3}$$

$$\Delta_u = -0.28s - 121$$

where s=spacing, in millimeters, of the fasteners located on the panel exterior/perimeter. The equivalent yield force and ultimate deformation, each as a function of nail spacing, are presented in Figs. 7 and 8, respectively.

Substitution of these expressions into Eq. (1) provides a mechanistic damage model that is only a function of the nail spacing and nonlinear structural response. Mathematically, this can be expressed as

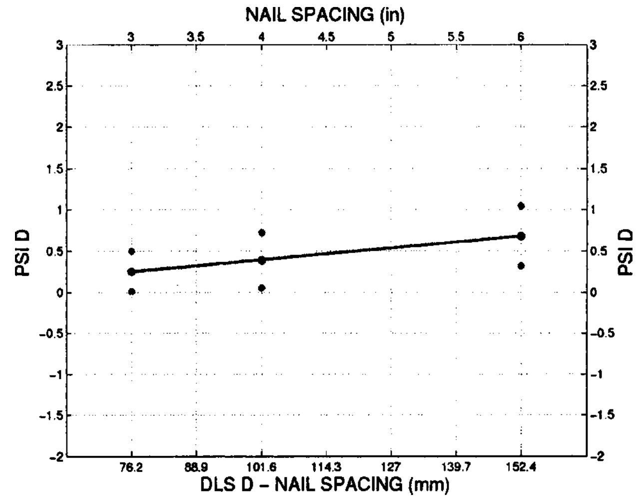

Fig. 6. Example of the ψ values calculated for limit state D

$$D = \frac{\Delta_m}{a_{\Delta_u}s + b_{\Delta_u}} + \frac{a_\psi s + b_\psi}{(a_{F_{ey}}s + b_{F_{ey}})(a_{\Delta_u}s + b_{\Delta_u})}\int dE \tag{4}$$

where a indicates the slope of the regression line and b=intercept with the ordinate.

Reliability Calculations

Seismic reliability analysis of woodframe structures is influenced by a large number of random variables. The most dominant having been shown to be the variability in the earthquake load. The resistance, or structural, side also has variables that affect the reliability such as panel grade and thickness, stud specific gravity, fastener type and size, anchorage, fastener spacing, etc. As explained earlier, fastener spacing was selected for illustrative purposes because it has been shown to significantly affect the response and is a convenient design parameter used in shearwall design. Existing software (Folz and Filiatrault 2000, 2001a) was used to model dynamic behavior of all the wood shearwalls within the structure. Suites of ground motions for Los Angeles were scaled and served to provide the load variability for the reliability analysis similar to the approaches used by Rosowsky (2002) and van de Lindt and Walz (2003). The damage indices for each wall were computed from the response time histories for each shearwall within the structural model using the mechanistic damage model that was calibrated herein. The damage indices were then combined to estimate damage to the entire structure. The model used for this example was a rectangular 4.88 ×9.74 m (16×32 ft) one-story building. Each of the shorter sides has one shearwall with 76 mm (3 in.) spacing on the panel exterior and each of the longer sides has two shearwalls with 152 mm (6 in.) spacing on the panel exterior as shown in Fig. 9.

Table 1. Slope and y-Intercept Values for Linear Regression of ψ as a Function of Perimeter Nail Spacing

Limit state	Slope	Y intercept
A	−0.0025	1.29
B	−0.0115	3.05
C	0.0029	−0.17
D	0.0057	−0.18
E	0.0026	−0.18

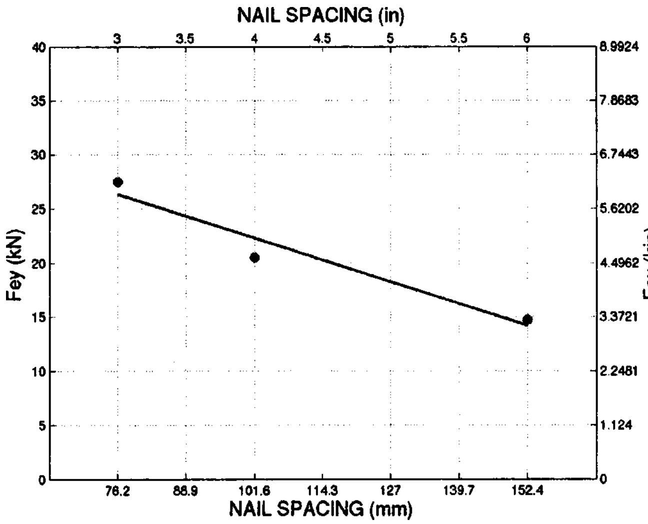

Fig. 7. Equivalent yield force, F_{ey}, as a function of nail spacing

The base shear, V_{base}, for the structure can be expressed as the product of the applicable portion of the weight, W, and a seismic response coefficient, C_s (ASCE 1998)

$$V_{\text{base}} = C_s W \tag{5}$$

where

$$C_s = \frac{S_{\text{DS}}}{R/I} \tag{6}$$

where S_{DS}=design spectral acceleration, R was set equal to 6.5 for wood shearwall structures, and the importance factor $I=1.0$. Assuming the fundamental period of vibration, $T=0.2$ s (Porter et al. 2001), one can calculate $C_s=0.21$ for Los Angeles. Using *Table 5-4 of the APA Panel Supplement to* AF&PA/ASCE (1996) and rewriting Eq. (5) in terms of mass, m

$$V_{\text{base}} = 0.21\ mg \tag{7}$$

For the city of Los Angeles one can set V_{base} equal to the design nominal shear capacity of the walls in the direction of interest and solve for the code maximum allowable seismic mass.

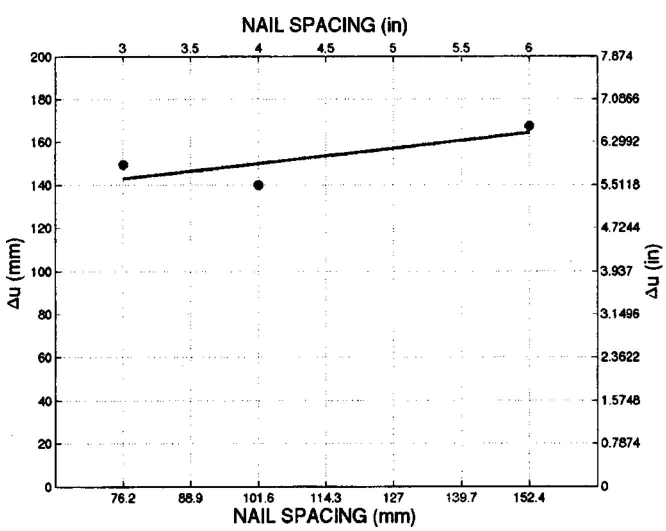

Fig. 8. Ultimate deformation, Δ_u, as a function of nail spacing

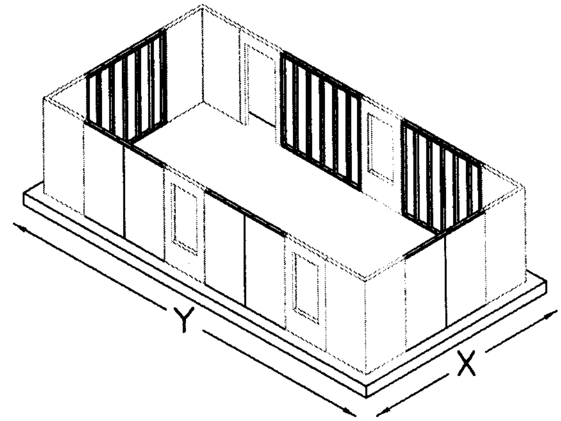

Fig. 9. Symmetric one-story building model used in the damage-based seismic reliability example

Using existing software developed as part of the CUREE-Caltech Woodframe Project (Folz and Filiatrault 200lb), it was possible to obtain the properties describing each of the shearwalls within the structural model. The fastener input used to obtain the shearwall parameters was obtained by Rosowsky and Kim (2001) using data from Dolan (1989); and Dolan and Madsen (1992a, b). They are for 8d-Common $63.5\times\varnothing 3.33$ mm (2-1/2 $\times\varnothing 0.131$ in.) nails used to attach 9.53 mm (3/8 in.) plywood to shortest path first dimension lumber framing. The resulting hysteresis favorably compared with reversed cyclic tests not discussed herein. This was the case even though the walls tested were not tested using the CUREE-Caltech test protocl, which is used by the existing software package mentioned above.

The ground motions used as input to the existing software consisted of twenty (20) SAC records (Somerville et al. 1997). Two different suites of earthquake records were used in the reliability analysis. The first suite consisted of 10 earthquake ground motions scaled to have a probability of exceedence of 2% in 50 years and the second suite consisted of 10 earthquake ground motions scaled to have a probability of exceedence of 10% in 50 years. The scaling was standard spectral scaling using ASCE (1998) 5% damped maps. A seismic analysis of woodframe structures program (Folz and Filiatrault 200lb) was used to obtain the hysteretic response of each of the six shearwalls in the single-story structure. The structure was analyzed by applying the ground motion in each direction. Because of the symmetry of the structure it was not necessary to compute the damage to the entire structure, i.e., it would be the same for each parallel side of the structure. However, the damage to an entire structure during an earthquake can be computed by weighting the individual assemblies (shearwalls) based on the hysteretic energy each wall dissipated during the earthquake. This has been expressed as

$$D_{\text{building}} = \sum (\lambda_i)_{\text{wall}} (D_i)_{\text{wall}} \tag{8}$$

where D_{building}=damage index for the building, $(D_i)_{\text{wall}}$=damage index for the ith shear wall, and $(\lambda_i)_{\text{wall}}$=weighting factor defined as

$$(\lambda_i)_{\text{wall}} = \frac{(E_i)_{\text{wall}}}{\sum (E_i)_{\text{wall}}} \tag{9}$$

where $(E_i)_{\text{wall}}$=total hysteretic energy dissipated by the ith shearwall. Once the damage indices were obtained, they were fit to a lognormal distribution and the data was rank-ordered for comparison with existing FEMA collapse prevention drift criteria.

The failure probability, P_f, can be read directly from the log-normal cumulative distribution function (CDFs) in Fig. 10. Limit state E was chosen here for discussion, and all the results are available in de Melo e Silva (2003). The P_f, i.e., at a damage level of unity for the 2% exceedance in 50 years earthquake suite (solid curve), can be found by following the vertical dashed line in the leftmost window of Fig. 10, intersecting the CDF, and reading the corresponding ordinate. Subtracting the ordinate value from unity gives the failure probability, which for the present example is approximately 0.05. This corresponds to a reliability index of 1.66. The FEMA collapse prevention limit state is 3% transient drift, whose P_f can be read from the rightmost window of Fig. 10 as approximately 0.12. This corresponds to a reliability index of 1.2. The dashed horizontal lines show the difference between these two failure probabilities which serves as a brief quantitative comparison. This difference could be used to back-calibrate basic structural (or nonstructural) behaviors to meet or improve upon FEMA drift provisions, although it should be pointed out that this is not at all a trivial calibration and would require extensive study and the use of probabilistic models for sensitivity analysis, etc. Nevertheless, the methodology presented herein could lend itself to this type of calibration. Although not conclusive because we are discussing a single simple symmetric structure, the concept of correlating some semblance or detail of structural behavior to one or more existing limit states is, at least, apparent from this illustrative example.

Summary and Conclusions

The concept of a seismic damage model for light-frame wood structures was discussed hypothetically and an illustrative calibration for a single design variable, perimeter fastener spacing, was demonstrated based on a fairly limited data set for a 2.44 $\times$ 2.44 m wood shearwall. Based on the procedure demonstration in this study, keeping in mind the limitations and assumptions, full development of a seismic damage model for woodframe structures appears to be possible. The example presented focused on limit-states that were more representative of severe structural damage, primarily because wall finish materials were not included. However, at present, knowledge of nonstructural element contributions and damage to them is the primary barrier to full development of such a methodology at the nonstructural damage level. Seismic damage models at the assembly level or even the global (whole-structure) level could provide woodframe performance-based design code developers a significant amount of flexibility, particularly when correlating observed structural behaviors with drift-based performance provisions. A concept such as the one presented here could also provide a framework for confirmation or improvement on existing FEMA drift provisions for woodframe structures.

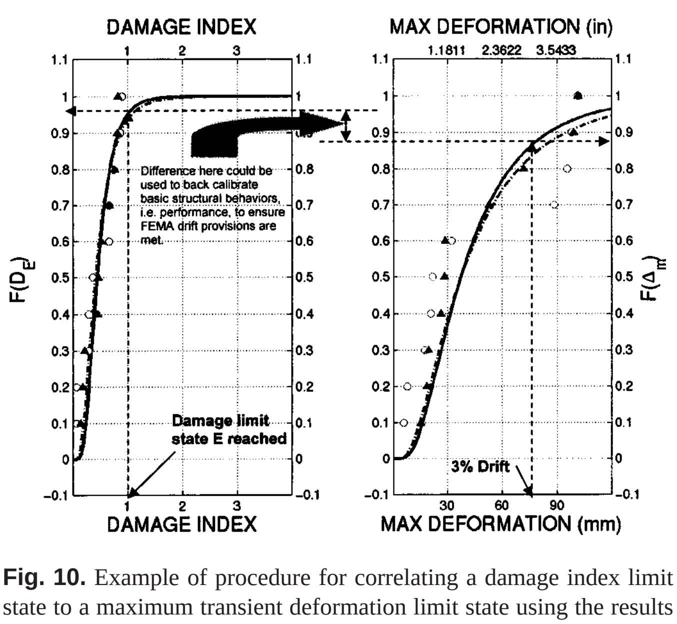

Fig. 10. Example of procedure for correlating a damage index limit state to a maximum transient deformation limit state using the results from nonlinear dynamic analyses combined with an earthquake record suite

Acknowledgments

The writer would like to acknowledge the partial support of the Federal Emergency Management Agency through the Michigan State Police Hazard Mitigation Grant No. FEMA-MI-1346. The writer would also like to thank Henrique "Kiko" de Melo e Silva for performing the wood shearwall tests, and also extends his gratitude to the anonymous reviewers for their suggestions for improving the final version of this paper

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