Damage and Damage Prediction for Wood Shearwalls Subjected to Simulated Earthquake Loads

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Abstract: Woodframe structures represent the most common structure type within the residential building stock in the United States. These relatively light structures perform well with regard to life safety and collapse during earthquakes, but can be significantly damaged resulting in large financial losses. Societal demands for damage-limiting design philosophies in the wake of the Northridge earthquake have fueled researchers (and practitioners) need to understand and better predict damage to woodframe structures. This paper examines damage to the lateral load carrying assemblies within woodframe structures, namely shearwalls. This is presented within the context of damage prediction for a wood shearwall assembly, and shortcomings and needs of such an approach are subsequently addressed. The incremental dynamic analysis approach is also examined as a possible tool for damage prediction. Qualitative damage descriptions and seismic force demands matched very well while maximum transient drifts did not match experimental results well. The potential for development of a whole-structure predictive damage model and its integration into the development of a performance-based seismic design development for woodframe structures is examined.

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CE Database subject headings: Shear walls; Earthquakes; Nonlinear response; Damage; Predictions; Dynamic analysis; Wood structures.

Introduction

Woodframe structures in the United States have performed well with regard to life safety and collapse during earthquakes. However, the level of damage sustained by woodframe structures during many of these same earthquakes was not felt by many (in both the public and private sectors) to be acceptable. Specifically, the 1994 Northridge earthquake caused a large amount of damage to woodframe structures [$9.6 billion in insurance claims (Kircher et al. 1997)], many built after the development and refinement of modern strength-based seismic design codes were in use. One result of this significant damage to woodframe structures was the Federal Emergency Management Association (FEMA)- funded “CUREE-Caltech woodframe project” whose results are beginning the process of being implemented. Another, perhaps eventually to become seminal, result was the motivation for seismic woodframe researchers to develop a performance-based seismic design (PBSD) philosophy for woodframe structures. Such philosophies are currently being developed for heavier materials such as steel and concrete, with the woodframe research community attempting to follow suit, but lagging behind. One reason for this lag is that the cost of a single woodframe structure is only a fraction of the cost of a typical steel or concrete structure, making structure/site specific dynamic (or performance) analysis too expensive for most potential homeowners. However, the number of buildings made up of these heavier materials is only a fraction of the number of woodframe structures, thus justification for a damage-limiting, or performance-based, seismic design philosophy is certainly warranted based on potential loss estimates for woodframe structures compared to these heavier and larger structures. It can be deduced that such a PBSD philosophy for light-frame wood structures may be unable to follow the exact path taken by PBSD developers focusing on these heavier materials. All PBSD philosophies seek to satisfy multiple performance criteria, one being a damage limiting criteria. Accurate damage prediction is one key to the development of such a design philosophy, and has not been addressed for woodframe construction.

A performance-based seismic design philosophy has yet to be undertaken for woodframe structures. However, some preliminary work has been performed by a handful of researchers. A direct displacement method was proposed by Filiatrault and Folz (2002) and allowable seismic mass charts for wood shearwalls based on FEMA drift limits were developed by Rosowsky (2002). One might expect that structural reliability will play a major role in the development of a PBSD philosophy for woodframe structures hence much of the initial work has focused on estimating reliability indices for woodframe structures and wood shearwalls. A study by van de Lindt and Walz (2003) developed and applied a new hysteretic model to estimate deformation-based reliabilities in Boston, Seattle, and Los Angeles. Fragility approaches (Rosowsky and Ellingwood 2002) have also been pursued recently and are the focus of ongoing work. A study by van de Lindt (2005) examines the possibility of using a mechanistic damage model for seismic reliability assessment of woodframe structures.
A more comprehensive summary of wood shearwall reliability studies prior to 2002 can be found in van de Lindt (2004)

The remainder of this paper presents the results of a study to examine the accuracy of “blind” damage predictions for wood shearwalls with the intent of possible integration into one or more areas of a PBSD philosophy, should the necessary extension and generalization to whole woodframe structures be possible. Damage predictions performed at Colorado State University (CSU) using a simple calibration (de Melo e Silva 2003) of the conceptual model developed by van de Lindt (2005) are compared to damage observations recorded during dynamic shearwall tests at Oregon State University (OSU) (White 2005). More specifically, incremental dynamic analysis (IDA) was used to determine what (earthquake specific) spectral accelerations will cause specific damage to a 2.44×2.44 m wood shearwall based on a Park–Ang damage measure (DM). Qualitative damage descriptions prepared independently by the CSU and OSU research teams are compared and discussed. It should be mentioned that there was no communication between the two research teams, other than the nominal shearwall design, the names of the earthquake records, and the peak ground acceleration (PGA) for each record.

Damage to Wood Shearwalls

Generally speaking, some level of woodframe damage has been observed during virtually all moderate to severe earthquakes. A significant amount of damage to wood shearwalls occurred during the 1994 Northridge earthquake and according to Schierle (2003) “besides building replacement and general earthquake repairs, shearwalls are the most expensive items to repair/replace.” Approximately 3% of all single-family and multifamily homes in Schierle’s damage statistic study (part of the “CUREE-Caltech woodframe project”) following the Northridge earthquake had shearwall damage (32 homes out of 1,129 included in the survey).

Surprisingly, the vast majority of woodframe earthquake research has focused on earthquake response, with the extension to damage receiving little attention with the exception of gathering loss estimates. It is one intent of this paper to investigate the possibility, albeit through several assumptions and discrepancies, that predictive mechanistic damage models may be able to play a role in emerging PBSD philosophies for woodframe structures.

Experimental Study for Seattle Earthquake Records

Fig. 1 presents a flowchart showing the procedure used for damage prediction and comparison between the OSU experimental study and the CSU predictive study. The dashed line indicates the portion of the chart for testing which is described below.

Test Setup

All walls were tested at the Gene D. Knudson Wood Engineering Laboratory in the Department of Wood Science and Engineering at Oregon State University. The load frame and detailed test setup is shown in Fig. 2. The bottom rail was a 102×152×10 mm steel beam that provided a moveable foundation for the shearwall. The beam was supported on pin joints that were connected to linear bearings at both ends. The linear bearings traveled on 51 mm steel shafts that were mounted to the strong floor of the lab. A 44.5 kN servo controlled hydraulic actuator with a 153 mm

Fig. 1. Flowchart depicting approach used to predict damage and qualitatively compare recorded and predicted damage

total stroke was attached directly to the foundation beam with a 90.0 kN dynamically rated in-line load cell. The inertial mass was placed on a cart and coupled to the wall by means of a lever assembly. Two 25.4×914×914 mm steel plates fastened to a four-wheeled cart provided the inertial mass for the system. A laterally braced tower supported a 102×152×10 mm steel beam that acted as a vertical moment arm to couple the mass cart and a steel channel (top rail) attached to the top of the wall. The moment arm provided a 1:9 equivalent mass ratio at the top of the wall. Horizontal struts connected the mass cart and top rail to the moment arm. A second 55.6 kN rated load cell was mounted in-line between the top rail and the horizontal strut at the top of the wall. Absolute position of the top of the wall was measured with a string potentiometer mounted between the strong wall and the top rail. A more detailed description of the test equipment is given in Seaders (2004).

Wood Shearwall Specimens

The shearwall specimens were designed based on the 2000 International Residential Code (ICC 2000) prescribed brace panel construction. All specimens were designed as 2.44×2.44 m sections with 38×89 mm studs at 610 mm on center. Two 32/16 APA rated in 1,220×2,440×11.1 mm structural oriented strand board (OSB) panels were installed vertically and fastened with 8d box nails (2.87×60.33 mm) at 152 mm on center around the edges of the panels and at 305 mm in the panel fields. On the opposite side of the wall, two 1,220×2,440×12.7 mm gypsum wallboards were installed vertically and fastened to the framing with 41 mm long Number 6 rough bugle drywall screws at 305 mm on center on the edges and in the fields of the panels. The test specimen had two 12.7 mm A307 anchor bolts at 305 mm from each end of the wall. Hold-downs were installed at the ends of the wall and bolted into double end studs. The double end studs were nailed together
using a 16d (2.87 × 60.33 mm) nail at 610 mm on center. Hold-
downs were anchored directly to the foundation using 15.9 mm
Grade 5 (A325) bolts.

Earthquake Records

Three earthquakes from the SAC steel project time history suite
(Somerville et al. 1997) were selected in order to test two walls
using each earthquake record. They were selected from the 10% probability of exceedance in 50 year suite and the charac-
teristics of the records are shown in Table 1. Recall that the records used in the SAC steel project were scaled from the original ground motions to match a weighted design spectrum at periods of interest for steel structures. Because steel structures typically have a much longer period than low-rise woodframe structures, it was necessary to rescale the records. A similar approach to that used in the SAC steel project was used to rescale the time histories so they would be representative of typical woodframe structures in North America. The result was a spectral scilne in the range of 0.1–0.3 s. This approach was used by Rosowsky (2002) and van de Lindt et al. (2005) to scale ground acceleration records for

Table 1. Description of Earthquakes Used in This Study [Adapted from White (2005)]

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>SE03</th>
<th>SE07</th>
<th>SE13</th>
</tr>
</thead>
<tbody>
<tr>
<td>EQ name</td>
<td>1984 Morgan Hill</td>
<td>1949 Olympia</td>
<td>1965 Seattle</td>
</tr>
<tr>
<td>Recording location</td>
<td>Gilroy, Calif.</td>
<td>Seattle Army Base, Seattle</td>
<td>Federal Office Building, Seattle</td>
</tr>
<tr>
<td>Time</td>
<td>April 24, 1984</td>
<td>April 13, 1949 (11:56 PST)</td>
<td>April 29, 1965 (07:28 PST)</td>
</tr>
<tr>
<td>Mechanism</td>
<td>Strike-slip</td>
<td>Subduction intraplate</td>
<td>Subduction intraplate</td>
</tr>
<tr>
<td>Dist. from epicenter (km)</td>
<td>15</td>
<td>80</td>
<td>61</td>
</tr>
<tr>
<td>Magnitude ((M_w))</td>
<td>6.2</td>
<td>6.5</td>
<td>7.1</td>
</tr>
<tr>
<td>Site (soil condition)</td>
<td>(S_d) (soil)</td>
<td>(S_d) (soil)</td>
<td>(S_d) (soil)</td>
</tr>
<tr>
<td>S.F. (SAC-Seattle)</td>
<td>0.5824</td>
<td>0.9598</td>
<td>0.7572</td>
</tr>
<tr>
<td>S.F. (Original-Seattle)</td>
<td>1.654</td>
<td>5.125</td>
<td>3.998</td>
</tr>
<tr>
<td>Peak accel. (g)</td>
<td>0.3862</td>
<td>0.2948</td>
<td>0.3693</td>
</tr>
<tr>
<td>Peak vel. (cm/s)</td>
<td>32.8</td>
<td>35.9</td>
<td>45.6</td>
</tr>
<tr>
<td>Peak disp. (cm)</td>
<td>8.2</td>
<td>9.0</td>
<td>8.1</td>
</tr>
<tr>
<td>Length (s)</td>
<td>60.00</td>
<td>66.72</td>
<td>74.16</td>
</tr>
<tr>
<td>Time step (s)</td>
<td>0.020</td>
<td>0.020</td>
<td>0.020</td>
</tr>
</tbody>
</table>

Table 2 shows the maximum load, \(P_{max}\), and maximum deflec-
tion, \(\delta_{max}\), at the top of the wall during the dynamic tests. A post-
test wall evaluation was conducted for each test to determine the overall condition of test specimens by recording failure type and location for the primary elements of the wall (studs, top and bottom plates, sheathing, and fasteners). The earthquake tests exhibited several failure modes, each of which involved failure of fasteners connecting the sheathing and the framing. These fastener failure modes were classified into five general categories as depicted in Fig. 3. These were: (1) edge breakout from the nails or wallboard screws; (2) nail pull-through; (3) nail pullout; (4) local-
ized crushing of the gypsum wallboard; and (5) screw fracture attaching the gypsum wallboard. A detailed damage description of

![Diagram of test setup for dynamic tests of wood shearwalls at OSU](image-url)
the walls following each dynamic test is given in Table 2. Full test results of these and other shearwalls can be found in Seaders (2004) and White (2005).

### Damage Model

Damage to structures during earthquakes has been studied for decades [see e.g., Suidan and Fubanks (1973); Park and Ang (1985); Jeong and Iwan (1988); Rahnama and Manuel (1996); van de Lindt and Goh (2004)]. There are basically two types of damage models that have been pursued. The first type are models that attempt to predict structural damage based on some type of ground motion intensity measure, e.g., arias intensity. The second type of model couples a (usually nonlinear hysteretic) structural model with measured or simulated ground motions and bases the damage prediction on the response. The latter type of damage

### Table 2. Damage Descriptions for Three Wood Shearwalls Prepared by OSU Team

<table>
<thead>
<tr>
<th>Earthquake record ID</th>
<th>$F_{\text{max}}$ (kN)</th>
<th>$\Delta_{\text{max}}$ (mm)</th>
<th>Damage description</th>
</tr>
</thead>
<tbody>
<tr>
<td>SE03</td>
<td>16.31</td>
<td>19.2</td>
<td>Some localized crushing of gypsum wallboard screws. Very little damage to fasteners attaching OSB sheathing to framing—slight nail withdrawal from framing members along panel edges (at most five nails) with a maximum of 3 mm withdrawal. Very little damage overall.</td>
</tr>
<tr>
<td>SE07</td>
<td>19.69</td>
<td>56.1</td>
<td>Edge breakout of gypsum wallboard screws that were along panel edges along bottom of wall. Some wallboard screws fractured along center stud of wall. Wallboard was basically connected to wall after the test by field nails. Nail withdrawal from framing (attaching OSB to studs). In some instances withdrawal was severe, generally between 5–25 mm and most predominant along panel edges and concentrated at the mid height of the wall—less severe along outer perimeter. Sheathing pulled away from the framing at variable amounts. Damage was about the same as SE13.</td>
</tr>
<tr>
<td>SE13</td>
<td>23.38</td>
<td>59.5</td>
<td>Edge breakout of gypsum wallboard screws along sill plate and vertical panel edges. Some fracture of wallboard screws—basically very little capacity remaining in wall. Nail withdrawal from framing members 5–25 mm—this was primarily seen along outer edges of wall, and along mid height of wall. Some nails were pulled through the sheathing—these were seen along the perimeter of the wall, and were few in number. Sheathing pulled away from the framing at a variable degree. Damage was about the same as SE07.</td>
</tr>
</tbody>
</table>

Fig. 3. Failure modes identified and recorded by OSU team following testing.
Mechanistic Damage Model

To date, only one mechanistic damage model has been attempted for woodframe structures (van de Lindt 2005). That model was discussed primarily in concept because the test calibration was performed for wood shearwalls with structural sheathing, but no finishing materials, meaning damage estimates of realistic finished structures were not possible because they have been shown to significantly effect the response of a woodframe structure (Flit atrault et al. 2002).

The model is based on the Park–Ang damage measure, $D$, which is expressed as

$$D = \frac{\Delta m}{\Delta u} + \frac{\psi}{F_{ey}} \Delta u \int dE$$

where $\Delta m$/maximum deformation during the earthquake; $\Delta u$/ultimate deformation (note that this is not the deformation corresponding to ultimate capacity but rather the largest deformation reached prior to failure) under monotonic loading determined experimentally; $F_{ey}$/yield force of the wall (which can be calculated as an equivalent yield force using a least-squares approach since wood shearwalls do not possess a distinct yield point); $\int dE$/incremental hysteretic energy absorbed by the wall during the earthquake; and $\psi$/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 $\Delta m$ 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 measure, $D$, where the value of $D=1$ signifies failure or, at least, that the predefined damage-based limit state has been reached.

The model was calibrated for bare 2.44×2.44 m wood shearwalls by dynamically testing a series of walls with varying sheathing panel perimeter nail spacing. The details of the experimental investigation can be found in de Melo e Silva (2003). The panel exterior nail spacing during those tests was varied between 75 mm (3 in.) and 150 mm (6 in.). All wall-specific parameters in Eq. (1), namely $\Delta u$, $F_{ey}$, and $\psi$ were linearly regressed on this nail spacing resulting in the expression

$$D = \frac{\Delta m}{a_0s + b_0} + \frac{a_0s + b_0}{(a_{ey}s + b_{ey})(d_{ey}s + b_{ey})} \int dE$$

where $a$ indicates the slope of the regression; $b$ indicates the $y$ intercept; and $s$=panel perimeter nail spacing. Two damage indicators (DIs) were selected for calibration and use in the present study: (1) corner nail pullout of the sheathing-to-framing nail (see Fig. 4) and (2) sheathing separation between the framing and sheathing material (see Fig. 4). No DIs were applied for the gypsum wallboard used in the OSU wall tests, because the original tests by de Melo e Silva (2003) used to calibrate the damage model did not have wallboard present. This apparent discrepancy will be discussed later. Table 3 shows the regression parameters for each DI to be inserted into Eq. (2) for damage prediction. As one can see, the only values required to estimate a damage measure (DM), $D$, is the maximum deformation and the hysteretic energy absorbed from a numerical simulation.

The dynamic response of the wood shearwalls were obtained using the SAWS computer program (Flit atrault and Folz 2002) which models each shearwall using the hysteresis model shown in

<table>
<thead>
<tr>
<th>Damage indicator (DI)</th>
<th>$a$</th>
<th>$b$</th>
<th>$a$</th>
<th>$b$</th>
<th>$a$</th>
<th>$b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corner nail pullout</td>
<td>-0.00252</td>
<td>1.29</td>
<td>0.282</td>
<td>121.3</td>
<td>-0.159</td>
<td>38.46</td>
</tr>
<tr>
<td>Sheathing separation</td>
<td>0.00262</td>
<td>-0.175</td>
<td>0.282</td>
<td>121.3</td>
<td>-0.159</td>
<td>38.46</td>
</tr>
</tbody>
</table>
Fig. 5. Hysteretic oscillator used in SAWS [Filiatrault and Folz (2002), ASCE]

Fig. 6. IDA curves using maximum transient drift

At this point in the predictive analysis it is important to point out several assumptions and discrepancies that existed between the calibrated damage model, the shearwalls tested at OSU, and the dynamic modeling procedure. The assumptions/discrepancies that were present are addressed in the following ways: (1) The dynamic test results used to originally calibrate the damage model were for wood shearwalls with studs spaced 406 mm (16 in.) on center, while the shearwalls tested at OSU had studs spaced at 610 mm (24 in.) on center. The discrepancy was neglected in the predictive analysis due to the fact that nails on the interior studs do not carry a significant amount of load. In fact some approximate deflection models neglect their contribution entirely (e.g., Easley et al. 1982). (2) The shearwalls tested at OSU had GWB on one side, while the shearwalls used to calibrate the damage model did not. The assumption was made that since the damage measure, $D$, is a function of the deformation and hysteretic energy dissipated, predictions can still be made about the damage on the OSB side of the shearwall if the responses are calculated using a model with GWB on one side. Thus, no predictions are offered for the GWB side of the shearwall. While these assumptions were made it is important to point out that the most critical design parameters, namely panel exterior nail spacing and nail type, were nominally identical.

**Damage Description Based on IDA**

In order to predict the damage caused by the earthquake records described earlier, one can apply a relatively new method known as an IDA (Vamvatsikos and Cornell 2001). For purposes of this study only a single-record IDA will be employed. To perform an IDA, one simply successively scales an earthquake record based on spectral scaling (or simply peak ground acceleration) and then applies each scaled record to the nonlinear structural model. The plot of the ground motion description, in terms of spectral acceleration or PGA, versus a structural quantity or DI is a single-record IDA curve. Vamvatsikos and Cornell (2001) define a single-record IDA study as “a dynamic analysis study of a given structural model parameterized by the level of the given ground motion time history.” As one can discern, the IDA is an acceleration and structural model specific curve, thus as with the damage model itself, some additional modeling uncertainty may be introduced into the prediction.

One can then, for example, determine what spectral acceleration produces a predefined level of damage based on the model. Quantities such as interstory drift have also been used, primarily for integration into existing performance-based seismic design and reliability calculations [see e.g., Cornell et al. (2000)], which typically use a transient drift “rule” to indicate failure.

**IDA Observations**

Fig. 6 shows IDA curves for the three earthquakes used in the experimental investigation. In this case the maximum transient drift at the top of the shearwall is plotted versus spectral acceleration. Recall that the earthquakes were scaled using common spectral scaling resulting in the PGAs shown in Table 5.

**Table 4. Hysteresis Parameters for OSB and GWB Portions of Layered Wood Shearwall Model**

<table>
<thead>
<tr>
<th>Hysteretic parameters</th>
<th>$K_0$ (kN/mm)</th>
<th>$R_1$</th>
<th>$R_2$</th>
<th>$R_3$</th>
<th>$R_4$</th>
<th>$F_0$ (kN)</th>
<th>$F_1$ (kN)</th>
<th>$\Delta_0$ (kN)</th>
<th>$\alpha$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>OSB$^a$</td>
<td>2.63</td>
<td>0.062</td>
<td>-0.062</td>
<td>1.27</td>
<td>0.035</td>
<td>20.42</td>
<td>3.78</td>
<td>37.82</td>
<td>0.76</td>
<td>1.09</td>
</tr>
<tr>
<td>GWB$^b$</td>
<td>0.455</td>
<td>0.029</td>
<td>-0.017</td>
<td>1.00</td>
<td>0.005</td>
<td>15.83</td>
<td>3.56</td>
<td>609.6</td>
<td>0.80</td>
<td>1.10</td>
</tr>
</tbody>
</table>

$^a$Parameters computed using CASHEW (Folz and Filiatrault 2001a,b) based on hysteretic parameters for an 8d box, gun driven nail.

$^b$Parameters from SAWS final report (Filiatrault and Folz 2002). Note that these parameters are for a shearwall with studs spaced 406 mm (16 in.) instead of 610 mm (24 in.).
objective of spectral scaling is to provide a commonality, i.e., reference, for different earthquakes in the design of a structure having a certain fundamental period of vibration. Interestingly, what we notice here is that the response of the linear oscillator during the spectral scaling procedure between 0.1 and 0.3 s requires a lower PGA for SE03 to produce the same average pseudoacceleration as SE07 and SE13 over that period range. However, when the spectral acceleration at exactly $T_n=0.2405$ s (the fundamental period of the walls being analyzed) is introduced to successively scale the earthquake acceleration records for the IDA curves, there is a significant discrepancy in their PGA. This is apparent in the third column of Table 5. This particular observation further highlights the structure (and acceleration) specific nature of IDA. In other words it should be noted that this is not a weakness within the IDA procedure, but simply demonstrates the effect of using “first mode” spectral scaling. The same type of variations exist in the distribution of PGAs for a site-specific suite of ground motions.

**Damage Prediction**

Based on the maximum transient drifts ($\Delta_{m}$) in Table 5 one can see that these are well below the 1% FEMA drift limit (24 mm) for serviceability-related damage. However, as was outlined earlier and has been observed for many decades, damage to structures during earthquakes is often due to the cycling effect. Thus, the damage model described in Eq. (1) can also be used to develop IDA curves. In order to predict the extent of the damage to the walls tested by the OSU team, one can follow the right side of the flowchart in Fig. 1. The nonlinear oscillator model in Fig. 5 is applied and the hysteresis recorded for a scaled earthquake record. The hysteresis is used as input to the unknown maximum deformation and energy dissipated in Eq. (2), knowing only the nominal design of the shearwall including the nail spacing. The DM is computed for each of the values plotted versus the spectral acceleration of the excitation earthquake. The process was repeated for an array of spectral acceleration values ranging from near zero to approximately 1.5g.

The IDA curves for SE03, SE07, and SE13 are presented in Figs. 7(a–c), respectively. Table 5 also presents the values of the DM for each limit state calculated by applying Eq. (2) to each computed hysteresis. For example, identifying the DM for corner nail pullout on Fig. 7(a) as 0.58 and tracing vertically to the solid IDA curve, one can see that the spectral acceleration value is approximately 0.57g, which matches the value in Table 5. Finally, in order to develop a qualitative damage description the amount of damage is assumed to vary according to a simple power law. This can be expressed as

$$\Delta_{\text{corner}} = \frac{[\text{DM}]_{\text{calculated}}}{[\text{DM}]_{\text{calibrated}}} \times (\Delta_{\text{DM}})$$

where $[\text{DM}]_{\text{calculated}}= value$ determined from the numerical hysteresis; $[\text{DM}]_{\text{calibrated}}= value$ used for calibration of the damage indicator (typically unity); $q=power$; and $\Delta_{\text{DM}}= value$ of the damage indicator used to calibrate the DM.

In the present analysis, nail pullout was defined as occurring somewhere between 6.3 mm (1/4 in.) and 12.7 mm (1/2 in.), based on observations made during the tests by de Melo e Silva (2003) thus $\Delta_{\text{DM}}$ was set equal to both these values in order to provide a range. Substituting $[\text{DM}]_{\text{calculated}}=0.58$ for the SE03 analysis from Table 5, $[\text{DM}]_{\text{calibrated}}=1.0$, $\Delta_{\text{DM}}=6.35$, 12.7 mm, and $q$ was set equal to 3 (based on experience), yields two values that serve as a range for the nail pullout, $\Delta_{\text{corner}}$: 1.24 mm (0.049 in.) $\leq\Delta_{\text{corner}} \leq 2.5$ mm (0.098 in.). Of course, there is some level of subjectivity at this stage, this 1.6 mm (1/16 in.) was selected as the prediction for corner nail pullout. Similarly, for sheathing separation the prediction was $\Delta_{\text{sheathing}}=0.013$ mm, so it was predicted that no sheathing separation occurred. It is important to note that although the DM approach was mechanistic, when the DM exceeded unity, the maximum response was incorporated based on experience since this was beyond the calibrated range.

Finally, comparing the rightmost column in Table 5 with the rightmost column in Table 2 one can see that the damage descriptions are adequate, but certainly not perfect. The SE03 predictions were by far the best, and the ranking of which earthquake record most severely damaged the walls was good. The OSU observations were that SE07 and SE13 damaged the walls approximately the same amount, while the CSU qualitative damage description

<table>
<thead>
<tr>
<th>Earthquake properties</th>
<th>Maximum response</th>
<th>Damage measure</th>
<th>Prediction summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Record ID</td>
<td>$S_g$ (g)</td>
<td>PGA (g)</td>
<td>$F_{\text{max}}$ (kN)</td>
</tr>
<tr>
<td>SE03</td>
<td>0.57</td>
<td>0.225</td>
<td>13.61</td>
</tr>
<tr>
<td>SE07</td>
<td>0.93</td>
<td>0.283</td>
<td>19.97</td>
</tr>
<tr>
<td>SE13</td>
<td>0.92</td>
<td>0.280</td>
<td>22.28</td>
</tr>
</tbody>
</table>
prediction was that SE07 was slightly more damaging, although the deformation numbers provided in the damage description for nail pullout and sheathing separation were the same. The force demands predicted by the oscillator model were very close for SE07 and SE13, and slightly low for SE03. The maximum deformation response predictions at the top of the wall were significantly lower than those observed. Deformation response predictions were calculated using the second degree of freedom (SDOF) hysteretic model shown in Fig. 5. This SDOF model does not explicitly model uplift, which results in a rigid body rotation of the wall but very little damage. This may be the reason that the oscillator model is underpredicting the response. In a forthcoming paper by van de Lindt, this issue will be addressed through the introduction of additional model parameters, accounting for uplift and other hysteretic details not currently accounted for with state-of-the-art woodframe modeling procedures.

Summary and Conclusions

Damage estimation is expected to play a central role in the development of a PBSD philosophy for woodframe structures. The potential of developing predictive numerical tools for wood shearwalls and eventually woodframe structures was investigated using a qualitative approach between two different research groups. The level of uncertainty within this approach was felt to be representative of the level of uncertainty that woodframe PBSD code developers will face shortly, particularly the use of one laboratory’s results and another’s analysis. Several conclusions can be arrived at as a result of this study and based on observations made during development and tentative application of the model. The approximate damage model provided enough insight into the behavior of the corner nail and sheathing separation at midheight to provide a reasonable damage prediction even in light of several assumptions and discrepancies, thus indicating that accurate damage predictions may be possible if more precise calibration is undertaken. More accuracy could be achieved if numerous earthquake records are used to calibrate the DM. Several calibration points [setting Eq. \( q \) equal to values between zero and unity] for each DI should be pursued, so that the value of \( q \) in Eq. (3) is known more precisely. IDA may be able to provide some insight into the damage levels that one can expect in a woodframe structure. However, it is envisioned that multiple-record IDA analyses coupled with seismic hazard curves will be needed to apply IDA for seismic reliability analysis and subsequently within the context of PBSD.

Mechanistic damage models for woodframe structures appear to be a reasonable concept, but the task of accurate calibration is daunting. The design details of woodframe structures make the number of design variables, and thus the number of dimensions in the regression, quite large. It may be possible to eliminate many of these variables through extensive sensitivity investigations. The possibility of predicting damage based only on numerical simulation results is inviting and should be pursued. The ability to predict damage, and subsequently losses, mechanistically would provide the key to systematically addressing design code deficiencies in existing design codes and provide a valuable tool for the development of new performance-based design codes for woodframe structures.

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