Summary

This paper reports on an examination of the hysteretic response of a wood shear wall evaluated via incremental dynamic analysis. In this numerical study, the actual cyclic response of a wood shear wall is represented by a single-degree-of-freedom (SDOF) hysteretic system. For comparative purposes, the force-displacement response of the wall is modeled using: 1) a fully nonlinear envelope curve with no definable yield point or 2) a tri-linear envelope curve, with a specified yield point. Whereas Model 1 more realistically reflects the response from test results, Model 2 has been adopted in some codified design procedures. For the numerical study, each hysteretic model is calibrated using available test data from a full-scale shear wall undergoing a cyclic testing protocol. In turn, each SDOF wall model is evaluated using incremental dynamic analysis (IDA) as the assessment method. From this analysis, IDA curves are generated displaying the damage measure of inter-story drift versus the intensity measure of spectral acceleration. The IDA curves generated using hysteretic models 1 and 2 are compared at various inter-story drift performance levels. Within the context of this study, final comments are made on the appropriateness of defining a yield point for the force-displacement response of wood shear walls.

1. Introduction

Shear walls are utilized as primary structural components in the lateral load resisting systems of light-frame wood structures. Given their importance, a significant research effort has been undertaken over the years to quantify their structural behavior. Within this body of research, there has been a particular focus on the seismic performance of wood shear walls. Experimental evaluations have typically involved the cyclic testing of full-scale walls [1-3]. Results from these experimental studies have been used to establish and refine codified seismic design procedures for wood shear walls. Common across all of these experimental studies is the specification of a yield load and/or yield displacement.

Given that the cyclic response of wood shear walls is non-linear under essentially all load levels, the identification of a yield point is problematic. It is commonly specified in order to establish the ductility of the wall. Currently, there is no consensus on the specification of the onset of yielding in wood shear walls [4].

The main objective of this study is to investigate the consequence of specifying a yield point in the numerical modeling of the hysteretic response of a wood shear wall. For comparative purposes, the force-displacement response of the wall is modeled in two different ways. First, with a fully nonlinear envelope curve with no definable yield point. Second, with a tri-linear envelope curve with a specified yield point. Each hysteretic model is calibrated to cyclic test data from a full-scale shear wall. In turn, each SDOF wall model is evaluated using incremental dynamic analysis (IDA) as the assessment tool. Comparing the IDA results allows conclusions to be drawn on the appropriateness of defining a yield point for the force-displacement response of wood shear walls.
2. Representative Shear Wall and Associated Test Data

For this study, shear wall test data from the CUREE-Caltech Woodframe Project was utilized [5]. The first-story shear wall, with a pedestrian opening, from the full-scale, two-story, woodframe shake table test house was chosen as the representative shear wall. The geometry of this wall and panel layout is shown in Fig. 1a. The wall was composed of 38 mm by 89 mm Douglas-fir framing members (with studs spaced 400 mm on center), sheathed with 9.5 mm oriented strand board and nailed with 8d (⌀2.9 mm) gun-driven box nails (spaced 150 mm on the panel edges and 300 mm on all interior studs). Hold-down anchors were attached at each end of the wall and both sides of the door opening. Complete construction details for the wall are given in [5].

This shear wall was subjected to the CUREE-Caltech Woodframe Project testing protocol shown in Fig. 1b [6]. The resulting lateral force versus top-of-wall displacement cyclic response of the wall is shown in Fig. 1c. This cyclic response diagram exhibits features typical of wood shear walls with nails as the sheathing-to-framing connectors: initial nonlinear force displacement response, pinched hysteretic behaviour along with strength and stiffness degradation. From the test data, the maximum load and corresponding displacement values \((F_u, \delta_u)\) in the pull and push directions are, respectively, \((46.8 \text{ kN, } 51.3 \text{ mm})\) and \((-42.9 \text{ kN, } -51.4 \text{ mm})\). It is of interest to note that these maxima are reached at just the one-third mark into the testing protocol. It is also noted that these displacements values at maximum load correspond to approximately a 2% inter-story drift level for the wall.

![Diagram](a)  ![Diagram](b)  ![Diagram](c)

Fig. 1: CUREE-Caltech Woodframe Project shear wall. (a) Elevation view, dimensions and panel layout. (b) Testing protocol. (c) Cyclic force-displacement curve from test data [5].
3. Hysteretic Modeling of Wood Shear Walls

As already noted, and as shown in Fig. 1c, the force-displacement response of a wood shear wall with nails as the sheathing-to-framing connectors is fully nonlinear under monotonic loading and exhibits pinched hysteretic behavior with strength and stiffness degradation under cyclic loading. The author has previously developed a general hysteretic model to capture the global cyclic response of wood shear walls [7]. This hysteretic model is defined in terms of a number of specified path-following rules.

Figure 2a shows the idealized force-displacement behavior of a wood shear wall under monotonic loading to failure, presented in terms of top-of-wall lateral force $F$ and corresponding racking displacement $\delta$. This monotonic response is modeled by the following nonlinear relationship:

$$
F = \begin{cases} 
\text{sgn}(\delta) \cdot \left( F_0 + r_1 K_0 \delta \right) \cdot \left[ 1 - \exp \left( -\frac{K_0}{F_0} \delta \right) \right], & \delta \leq \delta_u \\
\text{sgn}(\delta) \cdot F_u + r_2 K_0 \delta - \text{sgn}(\delta) \cdot \delta_u, & |\delta| < |\delta| \\
0, & |\delta| \geq |\delta| 
\end{cases} 
$$

[1]

As shown in Fig. 2a, the monotonic force-displacement response of a shear wall can be characterized by six physically identifiable parameters: $F_0$, $K_0$, $r_1$, $r_2$, $\delta_u$, and $\delta_f$.

Fig. 2: Modeling the force-displacement response of a wood shear wall. (a) Parameters defining the envelope curve. (b) Parameters defining the cyclic load paths. (Reproduced from [7])

The force-displacement response of this model under cyclic loading is shown in Fig. 2b. The basic path following rules associated with this model for cyclic loading is briefly outlined. In Fig. 2b, force-displacement paths OA and CD follow the monotonic envelope curve, as given by Eq. [1]. All other paths are assumed to exhibit a linear relationship between force and displacement. Unloading off the envelope curve follows a path such as AB with stiffness $r_2 K_0$. Under further unloading the response moves onto path BC, which has reduced stiffness $r_2 K_0$. The very low stiffness along this path captures the pinched hysteretic response displayed by wood shear walls under cyclic loading. Loading in the opposite direction for the first time forces the response onto the envelope curve CD. Unloading off this curve is along path DE, followed by a pinched response along path EF, which passes through the zero-displacement intercept $F_1$, with slope $r_1 K_0$. Continued re-loading follows path FG with degrading stiffness $K_p$, as given by
\[ K_p = K_0 \left( \frac{\delta_0}{\delta_{\text{max}}} \right)^\alpha \]  

with \( \delta_0 = \left( \frac{F_0}{K_0} \right) \) and \( \alpha \) a hysteretic model parameter which determines the degree of stiffness degradation. Note from Eq. [2] that \( K_p \) is a function of the previous loading history through the last unloading displacement \( \delta_{\text{un}} \) off the envelope curve (corresponding to point A in Fig. 2b), so that

\[ \delta_{\text{max}} = \beta \delta_{\text{un}} \]  

where \( \beta \) is another hysteretic model parameter. With this model, under continued cycling to the same displacement level, the force and energy dissipated per cycle is assumed to stabilize.

For subsequent comparative purposes, this model will be identified as hysteretic Model 1. The 10 parameters required to define this model are listed in Table 1. Values for these parameters can be obtained for a particular shear wall by fitting the model to cyclic test data. A simple spreadsheet program can perform this task. In turn, with the parameter values defined, the cyclic response of the wall can be represented as a SDOF hysteretic shear spring for subsequent dynamic analysis [7].

<table>
<thead>
<tr>
<th>Hysteretic Model</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_y ) (kN)</th>
<th>( F_1 ) (kN)</th>
<th>( \Delta_1 ) (mm)</th>
<th>( \alpha )</th>
<th>( \beta )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.80</td>
<td>0.041</td>
<td>-0.038</td>
<td>1.10</td>
<td>0.010</td>
<td>33.2</td>
<td>-</td>
<td>4.0</td>
<td>51.3</td>
<td>0.80</td>
<td>1.20</td>
</tr>
<tr>
<td>2</td>
<td>4.19</td>
<td>0.024</td>
<td>-0.53</td>
<td>1.52</td>
<td>0.014</td>
<td>-</td>
<td>40.3</td>
<td>4.0</td>
<td>51.3</td>
<td>0.91</td>
<td>1.20</td>
</tr>
</tbody>
</table>

Table 1 Hysteretic model parameters for the shear wall

This fitting procedure was applied to the CUREE-Caltech Woodframe Project shear wall introduced in Section 2. Table 1 lists the associated model parameters resulting from the fitting procedure. Figure 3a shows the cyclic response of the fitted hysteretic Model 1 superimposed on top of the actual test data. Good agreement is observed between hysteretic Model 1 and the test results. The capability of Model 1 is further verified in Fig. 3b, which compares the absorbed energy (defined as the area under the force-displacement hysteresis loops) from the wall test with that predicted by the model. Very good agreement is shown up to the halfway point in the loading protocol.

Fig. 3: Comparison of hysteretic Model 1 with test data. (a) Force-displacement cyclic response. (b) Absorbed energy of the shear wall over the testing protocol.

Hysteretic Model 2 incorporates a defined yield point and assumes a tri-linear envelope curve. As noted previously, a number of methods have been proposed to define the onset of yielding in wood shear walls. For this study, the yield load will be defined according to ASTM E2126 [8]. In this
testing standard, yielding is based on the concept of an equivalent energy elastic-plastic (EEEP) curve. The EEEP curve for the CUREE-Caltech Woodframe Project shear wall is shown in Fig. 4 superimposed on top of the envelope curve obtained from the test data (averaged over the pull and push directions). According to ASTM E2126, the EEEP curve establishes the yield load through the following equation:

$$F_y = K_e \cdot \left( \Delta_u - \sqrt{\Delta_u^2 - 2A / K_e} \right)$$  \[4\]

The variables $A$, $\Delta_u$, and $K_e$, specified in Eq. [4], are identified in Fig. 4. By this method, $F_y$ is 40.3 kN. Also, based on ASTM E2126, the initial wall stiffness $K_e$ is 4.19 kN/mm.

With $F_y$ defined, the tri-linear envelope curve for hysteretic Model 2 is as shown in Fig. 4. The paths-following rules used to define the cyclic response for Model 2 are the same as for Model 1.

![Fig. 4: Determination of the yield load for the shear wall based on ASTM E2126.](image)

Following from this work, Table 1 lists the fitted parameters for hysteretic Model 2. In turn, Fig. 5a shows the cyclic response of hysteretic Model 2 superimposed on top of the actual test data. A comparison between the test data and hysteretic Model 2 shows a greater degree of discrepancy than did Model 1 (see Fig. 3a). Model 2 is further evaluated by comparing the absorbed energy from the wall test with that predicted by the model, as shown in Fig. 3b. Again, the accuracy is not as good as that predicted by Model 1 (see Fig. 3b).

![Fig. 5: Comparison of hysteretic Model 2 with test data. (a) Force-displacement cyclic response. (b) Absorbed energy.](image)
Incremental dynamic analysis is an effective numerical procedure for assessing a structure’s capacity under seismic demands [9]. With this method, the structural model is subjected to a suite of ground motions, scaled to multiple levels of intensity while monitoring a corresponding structural response quantity. To this end, an Intensity Measure (IM) must be identified that appropriately scales the earthquake records. A commonly adopted IM is the 5% damped spectral acceleration at the first-mode fundamental period of the structure $T_1$, denoted by $S_a(T_1, 5\%)$. For each earthquake record and given IM, a corresponding Damage Measure (DM) is determined through a non-linear dynamic analysis of the structural model. Typically, the DM is chosen to be the maximum inter-story drift $\theta$ occurring within the structure. The dynamic analysis results, over the suite of ground motions, can be conveniently summarized on an IDA graph, which plots IM, along the ordinate axis, versus DM. Additionally, the results from an IDA can be evaluated within the context of performance-based assessment procedures. As an example, the DM values can be related to the target performance limits of Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP), as specified by FEMA 356 [10].

In this study, all of the non-linear dynamic analyses were performed using the author’s SASH1 program. SASH1 performs the Seismic Analysis of a SHear wall modeled as a 1-degree-of-freedom hysteretic oscillator. Both hysteretic Models 1 and 2 can be accommodated by the program. SASH1 has the option to perform a single dynamic analysis or an IDA. With the IDA option, for each ground motion, SASH1 automatically scales the IM over a user-specified range of values and records the corresponding DM values, plotting the resulting IDA curve. SASH1 also includes an energy balance check to ensure the numerical accuracy of each dynamic analysis.

<table>
<thead>
<tr>
<th>Earthquake Event / Year</th>
<th>Station</th>
<th>PGA (g)</th>
<th>$S_a(T_1 = 0.25s, 5%)$ (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Superstition Hills 1987</td>
<td>Brawley</td>
<td>0.116</td>
<td>0.205</td>
</tr>
<tr>
<td></td>
<td>El Centro Imperial County Center</td>
<td>0.258</td>
<td>0.461</td>
</tr>
<tr>
<td></td>
<td>Plaster City</td>
<td>0.186</td>
<td>0.438</td>
</tr>
<tr>
<td>Northridge 1994</td>
<td>Beverly Hills 14145 Mulhol</td>
<td>0.416</td>
<td>0.799</td>
</tr>
<tr>
<td></td>
<td>Canoga Park – Topanga Can</td>
<td>0.356</td>
<td>0.793</td>
</tr>
<tr>
<td></td>
<td>Glendale – Las Palmas</td>
<td>0.357</td>
<td>0.950</td>
</tr>
<tr>
<td></td>
<td>LA – Hollywood Storage</td>
<td>0.231</td>
<td>0.509</td>
</tr>
<tr>
<td></td>
<td>LA – North Faring Road</td>
<td>0.273</td>
<td>0.555</td>
</tr>
<tr>
<td></td>
<td>North Hollywood – Coldwater</td>
<td>0.271</td>
<td>0.649</td>
</tr>
<tr>
<td></td>
<td>Sunland – Mt Gleason Ave</td>
<td>0.157</td>
<td>0.381</td>
</tr>
<tr>
<td>Loma Prieta 1989</td>
<td>Capitola</td>
<td>0.529</td>
<td>1.517</td>
</tr>
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<td></td>
<td>Gilroy Array # 3</td>
<td>0.555</td>
<td>1.619</td>
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<tr>
<td></td>
<td>Gilroy Array # 4</td>
<td>0.417</td>
<td>0.752</td>
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<td></td>
<td>Gilroy Array # 7</td>
<td>0.226</td>
<td>0.573</td>
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<tr>
<td></td>
<td>Hollister Differential Array</td>
<td>0.279</td>
<td>0.678</td>
</tr>
<tr>
<td></td>
<td>Saratoga – West Valley</td>
<td>0.332</td>
<td>0.694</td>
</tr>
<tr>
<td>Cape Mendocino 1992</td>
<td>Fortuna Boulevard</td>
<td>0.116</td>
<td>0.219</td>
</tr>
<tr>
<td></td>
<td>Rio Dell Overpass</td>
<td>0.385</td>
<td>0.930</td>
</tr>
<tr>
<td>Landers 1992</td>
<td>Desert Hot Springs</td>
<td>0.154</td>
<td>0.378</td>
</tr>
<tr>
<td></td>
<td>Yermo Fire Station</td>
<td>0.152</td>
<td>0.508</td>
</tr>
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</table>
In this investigation, the CUREE-Caltech Woodframe Project shear wall is required to carry a seismic weight of 64 kN, which is approximately two-thirds of its design value. Using the initial wall stiffness of $K_e = 4.19$ kN/mm, as determined from ASTM E2126, the fundamental period of the wall is $T_1 = 0.25$ s. Equivalent viscous damping of 2% of critical is assumed for both models.

To perform the IDA study, a suite of 20 ground motions, used in the development of the CUREE-Caltech Testing Protocol, was selected [6]. Identification of the 20 earthquake records and the associated peak ground accelerations (PGA) are given in Table 2. All of these ground motions are recorded far enough from the fault rupture to be free of typical near-fault pulse characteristics. The specified values of $S_a(T_1,5\%)$ given in Table 2 are based on a fundamental period $T_1 = 0.25$ s for the wood shear wall.

The results from this IDA study, are summarized in Fig. 6. The IDA graphs for hysteretic Model 1 and Model 2 are shown, respectively, in Figs. 6a and 6b. For assessment purposes, the DM values corresponding to the FEMA 356 specification of IO, LS and CP performance levels are shown on these IDA graphs. As expected with this type of analysis, there is a fair degree of dispersion in the results. This is attributable to the inherent variability in the ground motion characteristics. Figure 6c compares the median IDA response of hysteretic Model 1 with that of Model 2. This graph clearly shows a marked level of difference between the two hysteretic models, particularly up to an inter-story drift level of about 1.5%. For example, if the target performance level is IO (or 1% inter-story drift), the difference in the predicted median $S_a(T_1,5\%)$ values is of the order of 20%.

Finally, a (nonparametric) fragility curve for the damage measure exceeding CP, is shown in Fig. 6d. Over essentially all probabilities levels, hysteretic Model 2 predicts a greater spectral acceleration than does Model 1 to exceed the performance state of collapse prevention. This implies that hysteretic Model 2 predictions are not conservative when referenced to Model 1 results.

Fig. 6: IDA results for the CUREE-Caltech Woodframe Project shear wall. (a) IDA graph for hysteretic Model 1. (b) IDA graph for Hysteretic Model 2. (c) Median IDA curves for hysteretic Models 1 and 2. (d) Fragility curve for the DM exceeding CP.
5. Conclusion

This study has investigated the consequences of specifying a yield point for a wood shear wall from a numerical modeling perspective. To this end, the actual cyclic response of a wood shear wall was modeled as a SDOF hysteretic system. For comparative purposes, the force-displacement response of the wall was modeled using: 1) a fully nonlinear envelope curve with no definable yield point or 2) a tri-linear envelope curve, with a specified yield point. Each hysteretic model was calibrated using available test data from a full-scale shear wall undergoing a cyclic testing protocol. In turn, each SDOF wall model was evaluated using incremental dynamic analysis as the assessment tool. The IDA curves generated using hysteretic models 1 and 2 were compared at various inter-story drift performance levels. Within this assessment framework, it was shown in a number of ways that the two hysteretic models predicted measurably different results. Given the limited scope of this study, further investigation on this topic is warranted.

6. References


